The President, 1961-62

The Institution welcomes as its President for the Session, 1961-62, Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., who takes office on the 5th October.

Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E.

Frank Robert Bullen was born in 1909 and was educated at Bancrofts School, Woodford, Essex, where he was captain of his house and won distinction at games, being awarded both cricket and Rugby colours.

In 1928, he joined John Mowlem & Co. Ltd. Public Works Contractors, as Assistant under Agreement to Mr. J. A. Ross under Sir Henry Japp. He studied at West Ham Technical College and under A. J. Anido, B.Sc., A.C.G.I. (Tutor) for the London University B.Sc.(Eng.). After taking his degree in 1932 he remained with John Mowlem to obtain practical engineering experience. This he gained on the Ford Motor Company's works at Dagenham, many piled foundations in and around London, jetty and wharf works on the river Thames and elsewhere, the renewal of the dock entrance at the Royal Docks in cofferdams and the Central Line extension to Ilford. During the war, Mr. Bullen was responsible for the design and construction of the civil and structural works of the Pluto scheme at Tilbury and for the construction of jetties, dolphins and landing hards along the South Coast.

He set up in practice as a Consulting Engineer in 1944, specialising in foundation works. He has been responsible for the Cast Houses of the Queen Furnaces at Scunthorpe, new workshops for Ashmore, Benson, Pease & Co., at Stockton, the structure of Binns Store, Middlesbrough and the extension to Grosvenor House, London. One of his latest responsibilities has been the special piled foundations of the new multi-storey car park at Audley Square.

Mr. Bullen's connexion with the Institution of Structural Engineers began in 1939, when he was elected an Associate-Member. He became a full Member in 1948 and was elected to the Council the following year. Since then he has served on the Standing Committees dealing with Finance, Membership, Literature, Bye-Laws, Legislation and Special Awards and he has also served on the Aluminium Development Association Research Scholarship Committee. He has held honorary office for a number of years and for the last three years has been a Vice-President. For several years he was Chairman of the Literature Committee and he has also been Chairman of the Membership Committee and the Finance and General Purposes Committee. He has represented the Institution on the Parliamentary and Scientific Committee and the Architects Registration Council.

Mr. Bullen has read a number of papers before the Institution, for two of which he was awarded the London Prize, namely, "Unusual Design for a Large Constructional Shop" (1952) and "Extensions to Constructional Shop: 170 feet and other long spans" (1959) For the former paper he was also awarded the Silver Medal of the Institution. He contributed an article on Institution Awards to the special Jubilee issue of *The Structural Engineer* which was published to mark the Fiftieth Anniversary of the Institution in 1958, and he read a paper on "Trends in the Future of Site Investigations" at the Fiftieth Anniversary Conference.

He is a Member of the Institution of Civil Engineers and in 1937 won the Charles Hawkesley Prize with his design for a Tidal Wharf. He is also a member of the Association of Consulting Engineers and a Fellow of the Royal Society of Arts.

In private life the President-Elect devotes much time and energy to movements connected with the welfare of youth and the Institution's younger members can be assured of his special sympathy and understanding. Mr. Bullen is already well known to members in London and in the Branches, where he has frequently read papers and which he has visited in an official capacity as Vice-President, and during his term of office he will undoubtedly make many new friends throughout the country. Members in all classes will wish him well and those who are responsible for conducting the work of the Institution will welcome the opportunity of serving under him, knowing that they may look forward to a year of further progress and expansion for the Institution.

Professional Practice and Drawing Office Procedure

by

L. R. Creasy, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E.

Summary

This paper reviews the economic and procedural effects on Structural practice of developments in present day circumstance. These have introduced a wide choice of structural forms accompanied by codified design rules and the mechanisation of construction techniques.

An assessment is made of the cost of providing specialist services for a particular project; of the relative factors to be considered in the selection of the appropriate basic forms for the scheme and of the advantages of independent judgment in making this choice.

It is considered that the limited resources of the community in respect of design engineers and technical staffs must be applied in the most efficient manner.

This is achieved, in principle, by concentrating on uniformity and repetition of detail at the expense of some increase in material but without significant increase in overall cost.

Techniques are indicated for the preparation of working details which achieve this effect with improvement in clarity of presentation particularly in reinforced concrete construction.

Introduction

The Practice of the Structural Engineer was born some 60 years ago with the general advent of the first framed buildings in steel and concrete. Owing to interruptions from two world wars effective practice has been concentrated, generally, into one period of two decades before the second war and one decade since. The clean division in time has produced two periods which are quite distinct and strongly marked by characteristics which are peculiar to each.

During the first period new ideas developed slowly and the Structural Engineer was faced with an easily forecast and comparatively conventional solution to every project. Framed structures were a comparatively new development and the design and the supervision of the larger projects was commonly entrusted to the specialist Engineer. It was a period of alternating slumps and booms in which the latter barely absorbed the available potential in technical staffs and site labour.

The post-war years have been one of very rapid development. For any given problem the Structural Engineer is offered a very wide choice of alternative solutions. The level of new construction has been high throughout and the demand for staff and labour has been insatiable.

Paper to be read before The Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 12th October, 1961, at 6 p.m.

In the second period, techniques of design are widely known and largely codified. There are many tangible improvements in construction techniques. Large contracting organisations offer new forms of contract and undertake on demand their own structural designs.

It is proposed to consider the impact of these new factors on our current practice.

Relative Responsibility

For a given project there will be many different forms of construction and lay-out of the framework which will satisfy the basic 'user' requirements of the structure. When the 'form and layout' have been chosen the development of the detailed design is largely governed by Codes of Practice.

Many of these alternative forms are sponsored by specialist contracting organisations. These groups include fully competent technical staffs who will provide the design and construction of the structural framework and, in many cases, the complete project as a 'package' deal.

In determining the appropriate 'form and layout' for a particular project there are generally three factors to be considered, namely; 'cost,' 'amenity' and 'speed of construction.' The relative importance of these three factors will be largely determined by the functional requirements of the client and will be special to each project.

The relation between these factors will largely determine the basic development of the project. It will decide the form of construction, the column module and the basic profiles of the frame.

It is not generally possible to give equal importance to all three factors simultaneously. Emphasis on any one will generally reduce the influence of the other.

If 'amenity' or 'speed of construction' are the principal requirements some 'cost penalty' will generally be incurred in the layout or form of construction. If 'cost' is of first importance some sacrifice must generally be made in other directions.

When the 'basic selection' has been made of the 'form and layout' for the project with due regard to the accepted balance of 'cost,' 'amenity' and 'speed' the major pattern of costs for the work has been determined. Only secondary cost advantages can be achieved by variation in detail.

The selection of the appropriate 'form and layout' is the 'key' problem of the project. It is one that requires a specialist and independent study and is not suitable, in principle, to leave to the competing influences of alternative pressure groups.

This aspect of 'basic selection' remains the overriding field of responsibility of the independent Consultant. The solution of these questions is largely locked up in un-codified form within the experience

of individual Engineers much as was the position with design processes during the early days of structural development. Some examples of study in this field have been published but there is a very considerable scope for further development.[1,2]

Consultant Costs

The technical complexities of producing a modern commercial or industrial structure are now so extensive that a considerable number of specialists are involved. In present day practice, generally, these specialists are compensated by a fee which must be included in the real cost of the project.

The several Professional Institutions lay down a basic scale of fees. These are generally assessed as a fraction of the cost of that portion of the work on which the specialist is engaged. The latter will represent a varying proportion of the overall cost of the project in different cases.

For purposes of final assessment it is convenient to express these several specialist fees as a percentage of the overall cost of the project. In a typical case they may be assessed as follows :

Architect	5 per cent
Structural Engineer	2½ per cent
Heating and Ventilating Engineer	2 per cent
Quantity Surveyor	3½ per cent
Total	13 per cent

The figure of 2½ per cent indicated in this sum for the Structural Engineer is roughly appropriate to a reinforced concrete framed building of medium size ; in the case of a steel framed building the proportion might reduce to 2 per cent. In this case, by convention, the difference is represented by shop details prepared by the contractor the cost of which is absorbed in that of the fabricated material.

In general terms these fees are payable by the client in all cases—in some form or other. Where the contractor is directly employed by the client to provide a full design in lieu of the specialist service the fee is added to the cost of work. The latter is then correspondingly higher than when the design and working drawings are prepared by the Consultant.

The basic fee received for the work is required to cover the cost of the draughtsmen's time and overheads. In many cases these are approximately divided in equal proportions.

On this assessment the total sum which can be allowed in wages to the drawing office for the design and detail of the framework for a particular project is between 1 per cent and 1½ per cent of the overall cost of the construction. This is a target which must be met to satisfy the fundamental economics of the service.

In the last decade building costs and wages have risen in approximately similar proportions. The basic percentage fee calculated on the overall cost of building has remained constant.

During the same period considerable technical advances have been made in design. These have also been usually accompanied by an increase in the time required for the preparation of calculations and drawings. In many cases these advances have introduced alternative forms of construction of relatively lower cost. These two factors have had the double effect of reducing the relative effective overall cost of a project on which the fee is based and increasing the outlay in staff salaries.

The balance of these opposing influences on the drawing office of a relatively reduced income and an increased expenditure have now reached an uncertain stability. In order to remain within economic limits three alternatives are available, namely :

To increase the percentage fee

To reduce wages

Or to apply the drawing office energy more efficiently.

This paper explores the possibilities of the last alternative.

Conservation of Effort

The fundamental requirement for the preparation of the design and working drawings for a structure are simple to state. It is required to make such calculations as are necessary to ensure the stability of the framework and to prepare instructions which will ensure the accurate fabrication and erection of the work in the shops and at the site.

This basic requirement will have many different interpretations depending on the structural form and type of contractual arrangements which are made for construction. The different cases will have this in common, that the extent of the design and drawing work must be sufficient for ' safety ' and ' clarity ' but not be so generous as to be un-economic in the expenditure of the draughtsman's time.

There are at least three characteristics of modern practice which can be applied to reduce the labour of preparing designs and details within the requirements of ' safety ' and ' clarity.'

The first refers to the greater confidence which can be placed in conventional designs borne of longer experience of their use. These design rules are largely based on the behaviour of small scale individual units. Full scale framed structures generally develop additional stiffness from composite action and show a greater load capacity in service than can be forecast from the behaviour of individual members. This allows the ironing-out of peak conditions in the design and encourages greater uniformity in the selection of members of the frame.

In the second place the larger influence on costs which is shown by the choice of ' form and layout ' reduces the relative cost effect of variations in detailing. The time required for preparing details, generally, will comprise some two-thirds of the total drawing office effort. There is thus considerable scope for reducing labour by simplifying both the detail itself and the method of presentation.

A further advantage is to be gained from greater site experience and improved techniques. This is particularly true in heavy basement construction and in reinforced concrete work generally.

Simplified and repetitive details of construction lead to more economic working at site. They extend the mechanisation of site processes and reduce the demands on manual operations requiring the attention of specialist technicians in short supply. Where this aspect is properly presented to the contractor some compensating ' cost-advantage ' is gained.

Simple details demand repetition of section and will usually require some increase in material. All extra material required in this way does not necessarily incur a ' cost-penalty.'

The general cost pattern is already determined before the details are commenced. The nett effect on the overall cost of the work of the excess of material which is required to simplify the details is usually quite marginal.

Much of the close detailing of former methods arose from concern for the safety of the structure should

errors occur in placing reinforcement. The stability of the structure is not over-sensitive in this respect and is remarkably resilient, generally, to minor errors of interpretation or execution.

Over-elaborate details presented with an excess of instruction often invite the error they try to avoid from the very complexity of their presentation. Some licence can be allowed to the site in matters of interpretation without effectively encroaching on the margin of safety.

These several arguments lead to the First Principle of Conservation of Drawing Office Effort, namely; " Reduce costs by the selection of ' form and layout ' and use additional material to simplify the details." The application of this principle is discussed in the following sections.

Budgetary Control

If the work of producing the structural design and drawings for a project is to be economically justified the operation must be carefully planned within a budget.

It has been stated, for example, that the fee for the structural design and drawings for a reinforced concrete framed building amounts to 2½ per cent of the overall cost of the work. The Engineer's expenses for the work comprise Drawing Office time and overheads. These are commonly divided in approximately equal proportions.

On this basis the nett cost of the Drawing Office time must not exceed an ' allowable percentage ' of 1¼ per cent of the overall cost of a reinforced concrete project. This latter sum becomes a ' target ' of ' D.O.-cost ' for the project which must not be exceeded without loss to the organisation.

An alternative is to adopt a ' target ' of ' D.O.-time.' This is expressed in the number of man-weeks of staff time which may be allowed for the work.

A convenient unit for assessment of time is the ' work-value ' of the staff, defined as the average equivalent value of overall cost of work represented by each man-week of ' D.O.-time.' The ' staff work-value ' is calculated by dividing the ' allowable percentage ' into the average weekly salary of the staff.

The ' target ' of ' D.O.-time ' in man-weeks which can be allowed for the project is then deduced by dividing the ' work-value ' of the staff into the overall cost of the project. This method is probably of greater visual impact on the Drawing Office staff concerned.

For example : if the average staff salary is £1,100 per year the ' staff-work-value ' is £1,700 per man-week. One man-week of D.O.-time can be allowed for each £1,700 in overall cost of the work. For a job valued at £170,000 in overall cost, a ' target ' of 100 man-weeks of ' D.O.-time ' can be allowed for its development.

In the case of a steel frame structure the basic design fee is reduced to 2 per cent. On the same basis the staff ' work-value ' would be £2,100 per man-week for a steel frame project.

Whichever target is adopted it is important to make an assessment at the outset of the number of drawings required for the job. This provides a useful check on the ' work-value ' of each drawing and the average time which can be allowed each drawing.

The ' work-value ' per drawing is calculated by dividing the number of drawings into the overall cost of the project. The average number of man-weeks which can be allowed per drawing is then deduced by dividing the ' staff-work-value ' into the work-value per drawing. It will be noted that this assessment assumes that the time spent on design and preparation of bar schedules is included in the time allowed for the drawings.

The ' work-value ' of each drawing will vary with the type and complexity of the project between quite wide limits. For a small reinforced concrete framed building each drawing might represent only £3,000 in overall cost of work. For a multi-storey flush slab structure ' efficiently ' detailed, the ' work-value ' per drawing can be increased to more than £10,000.

On the basis indicated above, these two limiting values represent an average of 1·8 and 6·0 man-weeks of ' D.O.-time.' In practice, generally, the lower figure is outside the economic limits.

By means of these factors the progress of Drawing Office expenditure on the project can be assessed week by week. A check on ' solvency ' can be made at any time by comparing the target with the estimated fraction of the D.O. work then completed.

It is, perhaps, worth emphasising that this procedure is not intended to convert a Structural Drawing Office into an automaton. The ' targets ' are flexible and can only be used as approximations, but they provide an incentive for planning and help to eliminate waste.

Programming of Work

The development of a structural project is an exercise in co-operation between diverse groups of specialists and members of different professions. The different groups will include with the Structural Engineer, professional associates working in complementary mediums such as the Architect or Equipment Engineer. Each is dependent on the other for information which will enable him to complete his own section of the work.

This situation is fruitful in potential argument as to the responsibility for delays. Without careful programming the counter-play of demand for interdependent information can dissolve in frustration which grinds the work to a stop.

The most efficient method of working is to receive a fully cut-and-dried scheme from the client and to proceed in a vacuum at top speed to produce the structural design and working drawings. In practice this is not possible and some allowance must be made for the interruptions and change of mind which are part of the natural development of any project.

If the work of producing drawings is not to be hopelessly uneconomic, the Structural Engineer must plan the work in stages and endeavour as far as possible to ensure that each stage is cleared before the next proceeds. In this respect a broad distinction must be made between ' profiles ' and ' working details.'

The Structural Engineer must produce proposals for the ' profiles ' of the structural framework at a very early stage in the development of the project based on the Architect's sketch plans. These consist of the outlines of the beams, columns and slabs, together with the foundations so far as they affect the general planning.

The Structural Engineer must expect the ' profiles ' to pass to and fro between himself and his professional associates, with, generally, repeated modification. These drawings are the means by which the project is developed by all concerned and it cannot be expected that the best corporate solution will be hit-upon at the first attempt.

On this account it is not necessary to prepare detailed calculations at the outset of the design work. The profiles should be prepared on preliminary calculations which are subsequently confirmed if necessary. Re-

inforced concrete work is particularly convenient in this respect since wide adjustments in section modulus can be easily made within the same concrete profile by modification of the reinforcement.

When the profiles are settled, the details of the steel connexions and the reinforcement may be prepared. In theory there is a considerable advantage in linking the preparation of the details closely with that of the profiles. The problems are fresh in the mind of the designer and it is seemingly efficient to commit them to paper before the various complexities are lost.

This approach overlooks the basic practicalities. The profiles are closely knit with the interests of the allied professions and are subject to modification by them until fully incorporated in their own detail drawings.

It is not generally sufficient that the structural profiles shall be 'approved' by the other groups. It is essential that the structural details are not started until the profiles are fully 'incorporated' in their working drawings.

The reinforcement details are generally quite personal to the Structural Engineer. They may be efficiently developed to firm profile without affecting the work of the other groups. To avoid the wasted effort of alterations, they are best left until the last moment and until required by the site.

Simplified Design

In recent years the basic theory of structural design has very greatly expanded in both compass and complexity. The analysis of a modern structural frame from first principles can be a most time-consuming operation. The work of design must be reduced as far as possible, consistent with safety, by using devices of the type indicated below.

There are three stages in the process of preparing a structural design, namely, the calculation of the applied loads, the assessment of the induced bending moments and shear forces and the selection of a suitable section within allowable stresses.

The assessment of the loads and allowable stresses is now closely controlled by the Codes of Practice. In many cases the Codes have been incorporated in building Byelaws and have all the force of legislation.

The Codes provide some rule-of-thumb guides to the work of the second stage but considerable latitude is allowed the designer, generally, in the use of simplifying assumptions when calculating the induced bending moments in the frame. It is important that the maximum advantage is taken of these opportunities.

Where the structure is fully pin-ended the calculations are well defined and simple. In other cases the use of 'accurate' methods to determine the distribution of the moments can involve the expenditure of an immense amount of time which is not generally justified.

There will be many factors which are impossible to assess with great accuracy—the applied load, the rigidity of the joints and relative stiffness of the foundations. Except in special cases the use of constants and other rapid approximations usually provide an answer of sufficient accuracy which is consistent with the basic design conditions.

It is an essential requirement when using approximations that the frame design shall be 'consistent.' By which is implied that the relief of fixity at each beam support is carried into the column in constant magnitude and with corresponding allowance in the nett mid-span moment.

Provided 'consistent' allowances are made in this manner in the design the actual magnitude of individual restraint moments is not generally important. The forces may be expected to arrange themselves according to the condition of maximum resistance and the basic stability of the frame is assured.

This principle is the justification for the familiar procedure of adjusting the 'base-line' of the induced bending moments in the members of a frame. The device can be used to shorten the analysis, increase the degree of 'repetition' and simplify the detail.

A useful design aid is found in the application of the principle of 'membrane stability.' The lateral disturbing forces in the frame are carried by horizontal and vertical membrane action of the floors and walls to the ground. These assumptions allow the conversion into a pinned structure of a frame in which nominally fixed joints are otherwise required for stability.

The pin structure is generally more economic than the corresponding fully continuous frame both in cost of construction and in the expenditure of Drawing Office time. 'Membrane stability' can generally be provided at little or no extra cost to the structure.

In suitable application the computer can be employed as a most valuable design aid. In general use, the Engineer will employ only established computer programmes for the solution of a particular problem. Individual organisations will not generally be able to afford the cost of setting up a special programme for a particular problem.

Few computer programmes can 'design' in the same sense that the Engineer calculates loads, moments, and induced stresses and then selects a section. There is such a programme for multi-storey steel frames,[3] but most computer programmes require the loads and the sections of the members of the frame. From this information the computer calculates the induced moments and stresses.

A suitable established 'computer programme' can be used to effect very great economies in staff time. The Engineer will calculate the loads and make a very approximate estimate of the proportions of the members of the frame. The computer will reply with the stresses in the chosen sections from which adjustments can be made if necessary.

In such cases the cost of the computer is only that of the machine time, which is very low compared with the cost in the Drawing Office using approximate methods. An initial profile design is based largely on experience and very simple assessments. It matters little if the 'guess' is widely inaccurate. The computer will show-up the 'error.'

The most generally useful computer programmes in this respect are for the solution of steel portal frames of complex outline[4] and reinforced concrete grid frames[5] of the two-way spanning 'waffle' or 'diagrid' type.

The techniques outlined in this section require the team action of an experienced designer and supporting staff. The professional engineer will indicate profiles and simplifying assumptions based largely on experience. The supporting staff will produce a relatively elementary analysis using constants and factors.

The simplification of the design work may result in some increase in the material content of the work. This is consistent with the 'Principle of Conservation of Effort' already outlined.

Profile Drawings

These are the first link in the chain of communication with allied professions and of instruction to fabricating shop and site. They will consist, generally, of small-scale framing plans showing the sizes of the individual members and such additional diagrams as are required to show their relative position in plan and elevation.

This latter requirement is usually best met with small-scale cross sections in elevation through the framework. These are in any case required for a basement structure and to indicate structural walls.

It is of considerable advantage to the site and to the Structural Engineer's own staff to arrange the profiles as a set of self-contained outlines which clearly show the structural framework in isolation. This avoids the confusion which is caused where this information is pieced together from a number of associated and interlocking drawings many of which have been prepared by allied groups.

By this method some duplication of information on architectural and structural drawings is inevitable. A rigid discipline is required to reduce this to the essential minimum. Separate structural drawings are justified only so long as they are confined to the basic outlines of the framework.

The temptation must, therefore, be rigorously repressed to add walls and partitions with extensive volumes of such items as shading and hatching or to multiply lines and add details of connexions and reinforcement. These embellishments not only confuse the outlines on which major emphasis is required, but become a severe waste of effort when inevitable and necessary modifications are made. The structural profiles should more nearly resemble the 'stark skeleton' than the 'gilded lily'.

In a steel frame project it is convenient to add basic design data to the profiles such as end reactions, column loads and forces in the members of lattice frames. Where such a frame is cased the arrangement and sizes of the latter can be generally delegated to the Architect's drawings. The reinforcement of the casing may be indicated by a schedule or typical detail.

In a reinforced concrete project the site staff should be able to set up the formwork from the profile drawings and assess the required volume of concrete. The reinforcement as shown on other drawings is then placed within the forms erected. The fragment of a profile cross section is shown on Fig. 1. illustrating the scale and clarity of outline. The reference to sizes and dimensions must be related to the other drawings of the service.

It is efficient to prepare the profiles so that they can be developed from the preliminary stage to their final presentation as a construction drawing in a continuous process. Where much re-drawing seems inevitable it is alternatively possible during preliminary stages of development to indicate structural outlines on prints of the Architect's sketch plans. Record prints may be required for contract and other purposes at later stages, but re-drawing is most wasteful of Drawing Office energy.

When the profiles are reaching agreement it may be necessary to 'confirm' them with calculations. These are then based on firm information and can be made with the minimum of effort. Errors of 'undersize' arising from this check must, of course, be rectified but 'economies' can prove deceptive and should be avoided.

Reference has already been made to the manifold advantages of simplicity and repetition in outline and detail. Careful planning in the preparation of the profiles will save complications at site, avoid the risk of errors in interpretation and very considerably reduce the overall volume of subsequent detailing work required.

There is generally considerable scope for simplifying the arrangement of the foundations and basement structure. This is achieved with straight cutting lines and the minimum of variation in levels without miscellaneous projections and irregularities. Variations can be generally ironed-out by setting the profile to envelope conditions.

Simplification in the framework is achieved by reducing the range of different sizes of columns and beams. This encourages overall economies in construction from prefabrication of the formwork. Apparent economies from trimming the sections to a minor variation in load is quickly offset by increased costs of fabrication in shop and on site.

In a steel frame the universal sections are useful in this respect since they allow a variation in unit weight for the same size of section. A somewhat similar effect can be achieved in reinforced concrete by an adjustment of the steel reinforcement within identical sizes of member. This approach is discussed later when dealing with bar arrangements.

Working Details

These consist of details of the joint connexions and rod reinforcement. For reasons already discussed they should not, generally, be added to the profiles but included in separate drawings.

In preparing the detail drawings it is essential to avoid 'over-detailing.' This obscures the intention, prevents effective checking and leads to errors in the Drawing Office and on site.

In a steel frame project, it is not generally necessary to show more than representative details of the connexions of the members. These, with the schedule of design data as shown on the profile drawings, are generally sufficient for the contractor to prepare the workshop details.

A similar approach may be used for projects using proprietary floor or other types of specialist unit. In such cases the preparation of the details may be delegated to the contractor where the basic design data is clearly scheduled.

It is often claimed that the detailing of reinforced concrete is excessively time-consuming and uneconomic. The operation can be very considerably simplified by forethought and planning.

There are a number of devices which assist in this respect. Some of these are made at the expense of some extra material but with a corresponding gain in simplicity of detailing and of site working. Examples are discussed in the following sections.

Basic Steel Arrangements

Drawing Office work is reduced by planning the reinforcement to fall into a minimum number of simple basic patterns. These will be different for beams,

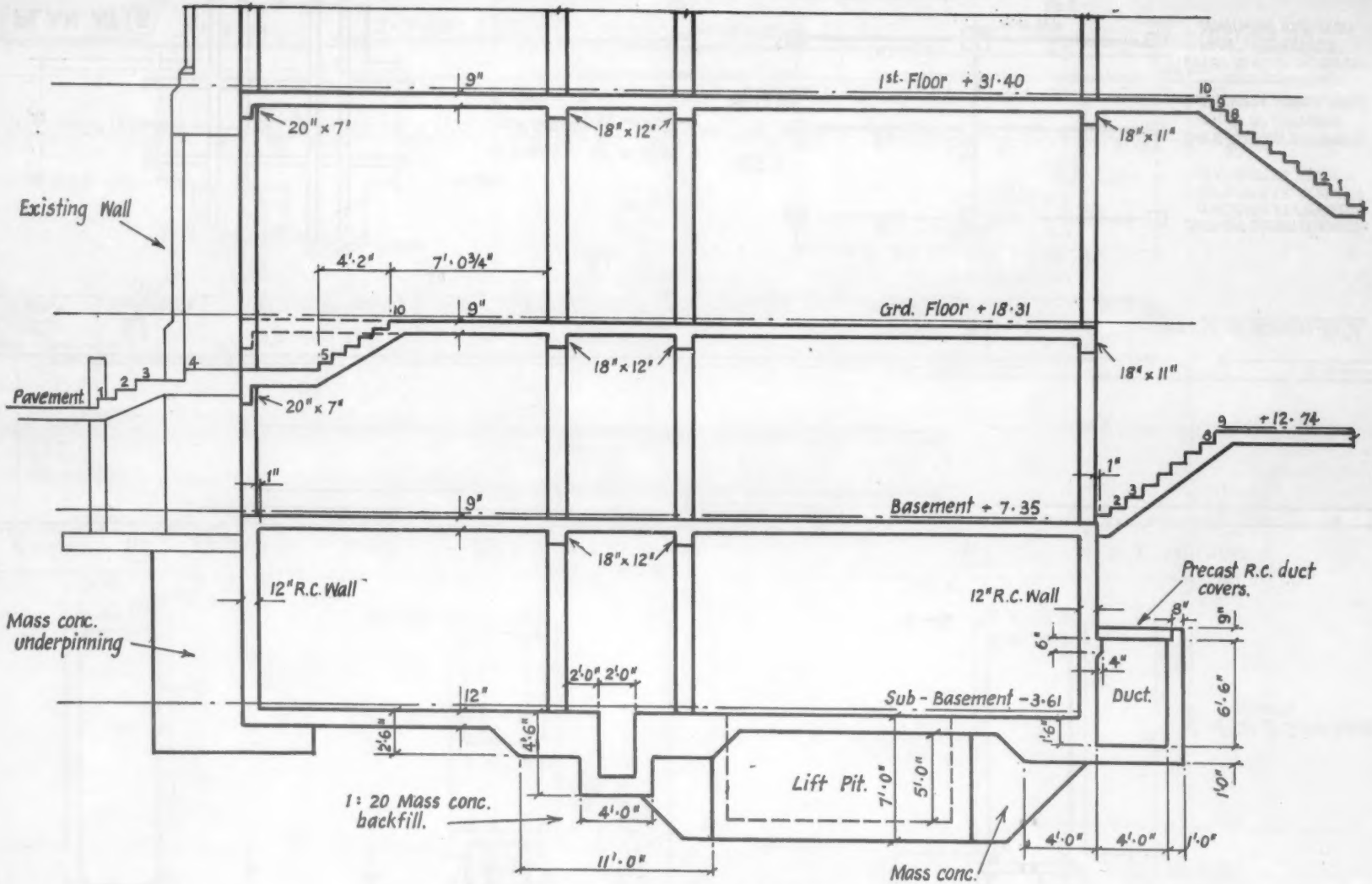


FIG. 1. PROFILE SECTION.

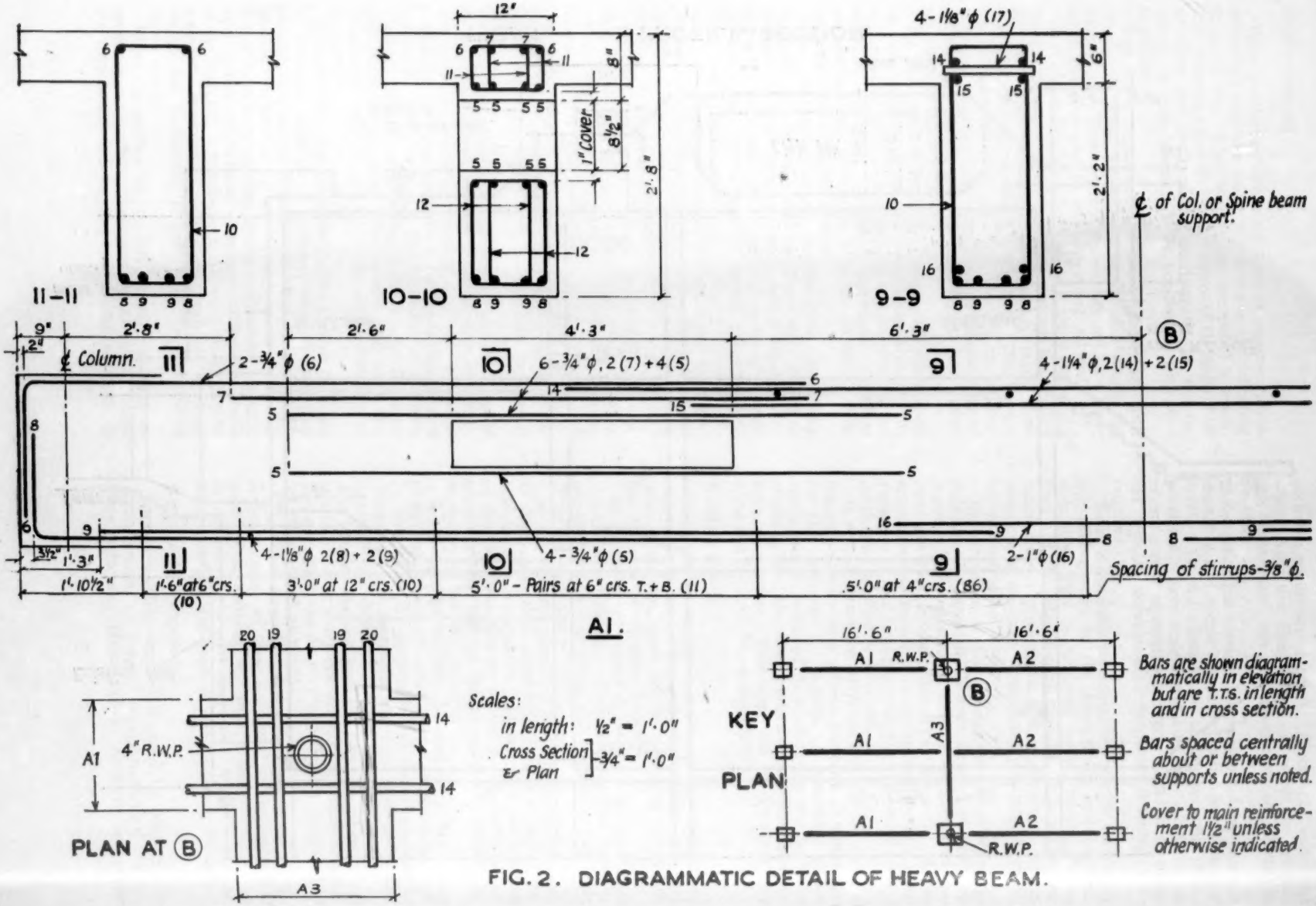


FIG. 2. DIAGRAMMATIC DETAIL OF HEAVY BEAM.

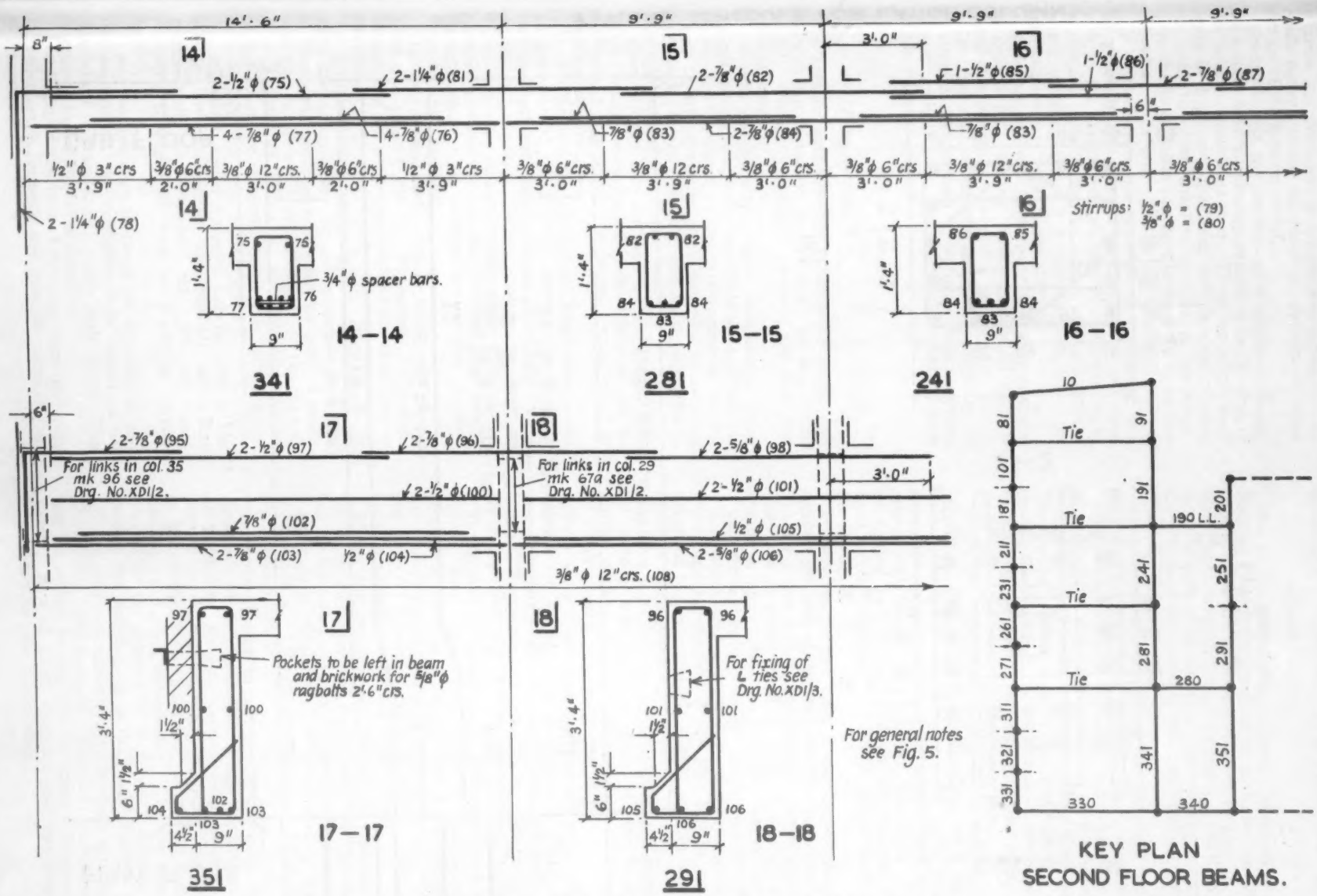


FIG. 3. DIAGRAMMATIC DETAILS OF LIGHTER BEAMS

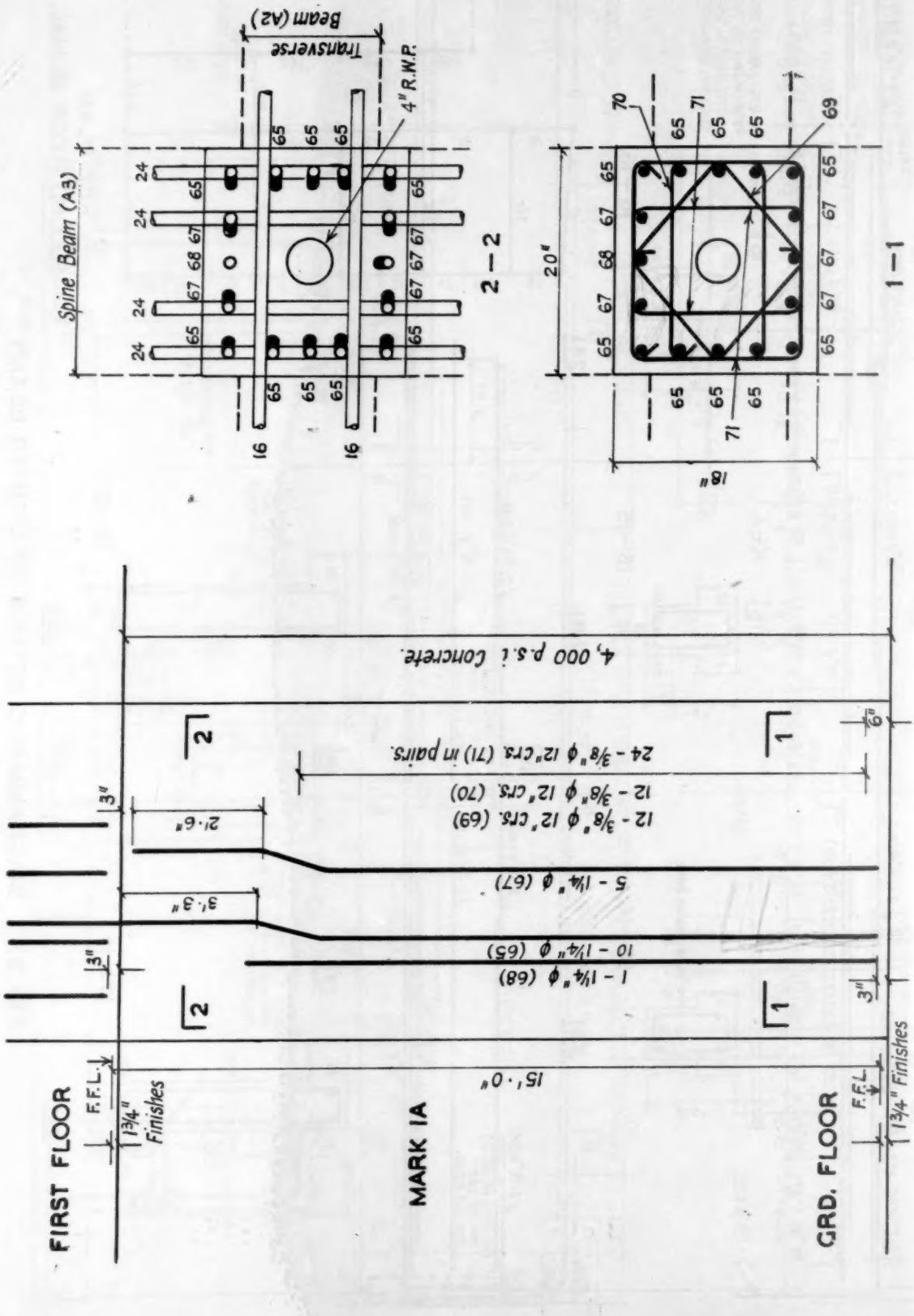


FIG. 4. TYPICAL SCHEDULE FOR LARGE COLUMN.

columns and slabs, but each basic arrangement of the reinforcement forms an identical pattern in cross section of the structural unit.

Graduations in the modulus of resistance of beams are achieved by introducing variations in the sizes of the bars and resistance arm. These variations must again be kept to a minimum and identical patterns are used to cover envelope conditions. In simple cases the basic forms can be scheduled by types and individual units ordered from the schedule.

As a corollary to this approach all reinforcing bars are generally laid straight save in exceptional cases such as the corner reinforcement to retaining walls. Hooks are replaced by the corresponding additional length of straight bar and bent-up bars for shear reinforcement are replaced by stirrups.

Basic patterns of reinforcement are easily arranged in general for columns. Reinforcement in slabs is based on a module of 6 in. pitch.

The repetition of pattern makes for simplicity of detailing and encourages repetition. It is surprising how complicated the detailing becomes when these simple rules are ignored.

Diagrams of Reinforcement

In conventional methods of detailing beams and columns it is customary to draw every bar and stirrup in full. These 'long-hand' methods are slow and often obscure.

The principal difficulty is that a vertical scale which is large enough to indicate clearly the relative position of the reinforcement is too cumbersome in length. There is also generally some difficulty in isolating individual bars for reference and in showing clearly the beginning and end of each bar.

Where straight bars are used without shear bends as previously recommended, these difficulties can be overcome by detailing the beam reinforcement in diagrammatic form.

In this method, the vertical and longitudinal scales are not identical. In elevation, bars are not drawn to scale in depth and may be shown out of position for clarity as necessary. Their position in length is drawn accurately to a convenient scale. The relative position between the top and bottom steel can be correctly shown and dimensioned in one or more 'true to scale' sections.

The longitudinal and vertical scales are usually $\frac{1}{4}$ inch and $\frac{1}{2}$ inch to the foot respectively. The shear stirrups can be written-up without drawing these in elevation.

It is useful to arrange the details of the beams one under the other, indicating the common centre lines of the columns or other basic reference lines. A small-scale plan is also useful on each sheet. In this manner a large group of beams is shown in semi-tabular form. Examples of beams detailed in the diagrammatic form are shown in Figures 2 and 3.

A somewhat similar approach can be used in the detailing of the columns. The bars are shown diagrammatically in elevation with 'true to scale' sections. Examples of these are shown in Figures 4 and 5.

It may be necessary to check the clearance of the reinforcement at the column-to-beam junctions for satisfactory clearances with larger scale plans. This is common to any system of detailing.

The diagrammatic form of detailing saves time from the reduction in linework and from the simplification of presentation.

Detailing by: 'Sections,' 'Type Plans' and 'Superimposed Details.'

One of the most time-consuming operations in the Drawing Office is the presentation of the wall and floor slabs. These are commonly detailed in plan or elevation by showing all the reinforcement in full throughout the length of the unit or alternatively by indicating the position of the bars by 'ticks' with only representative bars drawn in full.

In the conventional method the plans are further supplemented with some sections. Much cross-referencing is involved and it is always a problem to know whether the steel is 'written-up' on plan or section.

All this is very conducive to the making of mistakes. The detailer himself, is often the only man who can decipher the drawing; the intention is likewise obscure to the Engineer who is responsible for the work, and to the site staff who must interpret them.

Walls and slabs lend themselves to detailing by 'sections.' The required reinforcement for a simple unit of constant dimensions can be quite clearly shown by section only without duplicating the reinforcement in plan or elevation. The pitch of the reinforcing bars is indicated on the section. The total number required is a simple sub-division into the length.

In more complicated cases it may be necessary to split-up the area of the unit into identical zones and to show a new section each time the profile changes. The zone is dimensioned over which each specific section is applicable. An example illustrating the detailing of a stair by 'sections' is shown in Figure 6.

Transverse steel is specified by diameter and pitch. The actual sub-division of the long lengths can be delegated to the contractor, subject only to an indication of the required lap length.

In floor slabs, as in beams, simplification is achieved by avoiding bent-up bars. Negative steel is most simply arranged in 'mats' of steel bars. These can be prefabricated if the contractor wishes.

Floor slabs which span one way are conveniently detailed by 'sections' in this manner. Flush slabs as used in the modern office block present some complication. They commonly span in two directions and are sub-divided into column and mid-span strips.

This type of slab can often be efficiently detailed by 'type plans.' The reinforcement in one panel is detailed by plan as a 'type' with a distinguishing letter. A section may be required to show the relative position of the reinforcement in depth. Identical panels are referred to by the 'type' letter.

Where the reinforcement is strictly detailed to follow the minimum code requirements the variations become somewhat complex. Some excess of material is generally required to achieve a satisfactory degree of simplification in the bar arrangement. An example is shown in Figure 7.

The great obstacle to repetition is the many small variations which are introduced into the project by site conditions, planning, aesthetics or services. The simplest way of dealing with these is to detail them as self-contained 'irregularities' which are bonded into the main steel as necessary. The site may be referred to the Architect's or Service's drawing for position and modification to the profile if any.

Small holes, for example, may be dealt with as an 'irregularity' and trimmed. The main reinforcement may be moved to one side for small differences in centres. Bars of $\frac{5}{8}$ inch diameter and less may be cut in place on site if necessary.

FIG. 4. TYPICAL SCHEDULE FOR LARGE COLUMN.

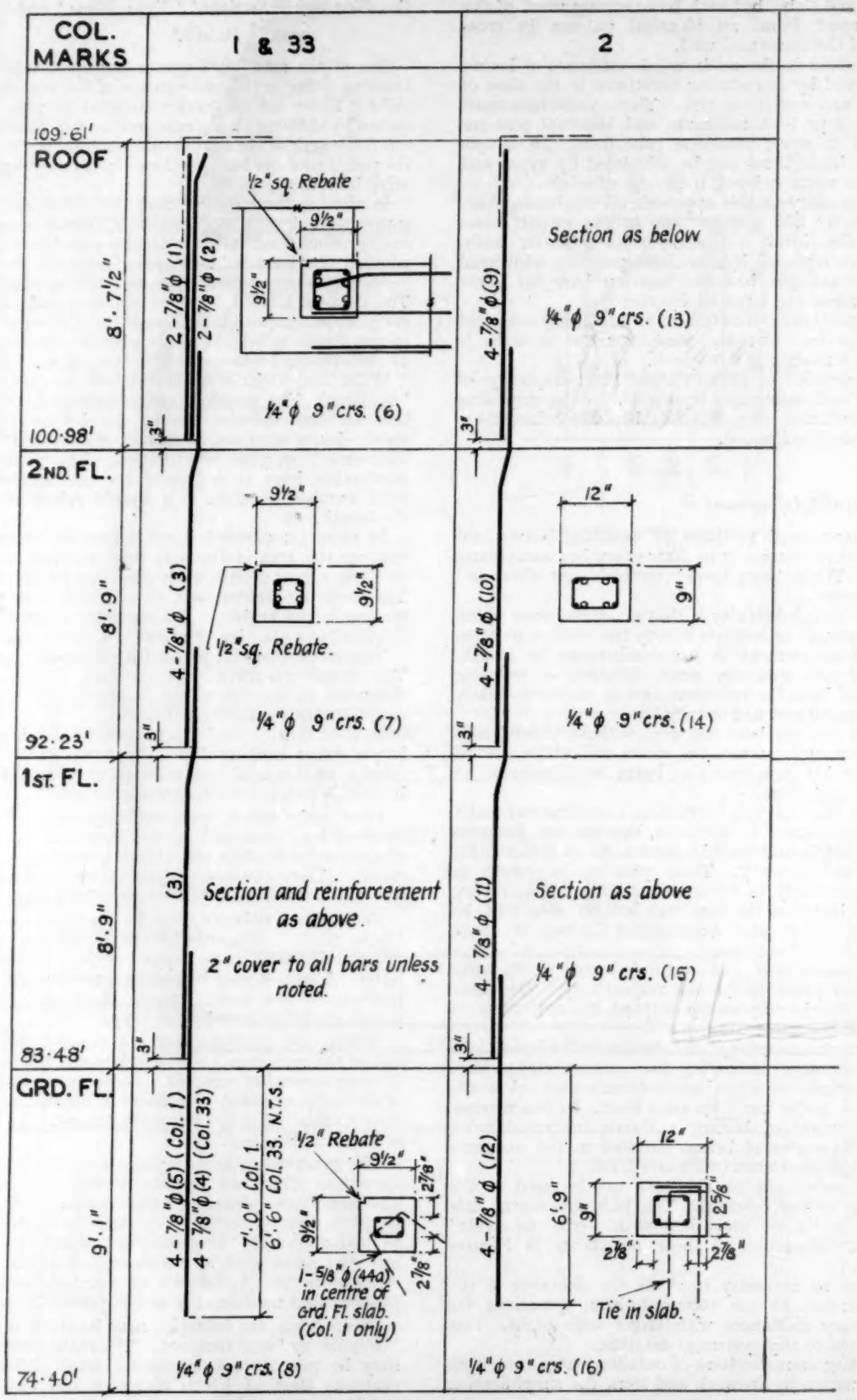
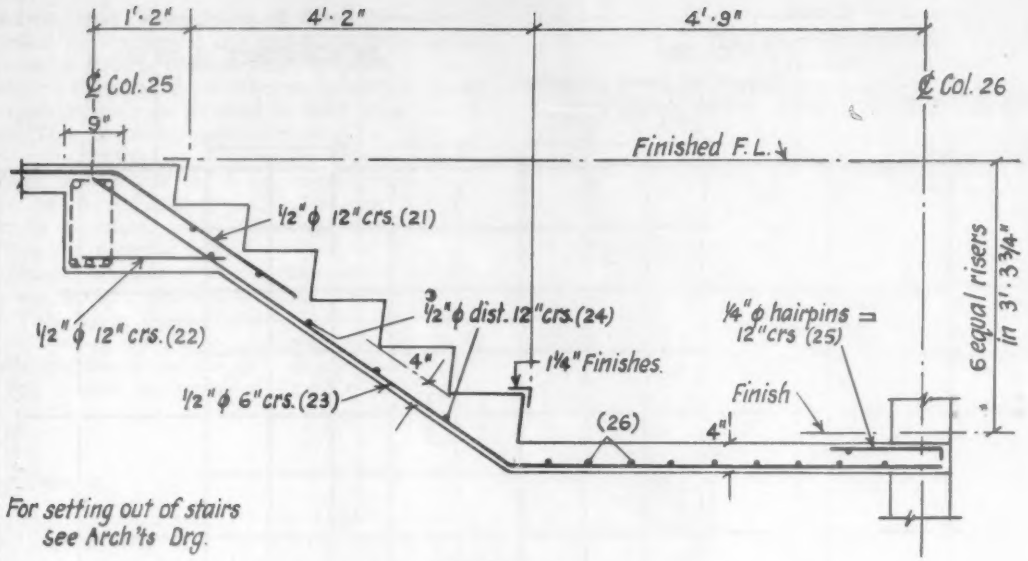
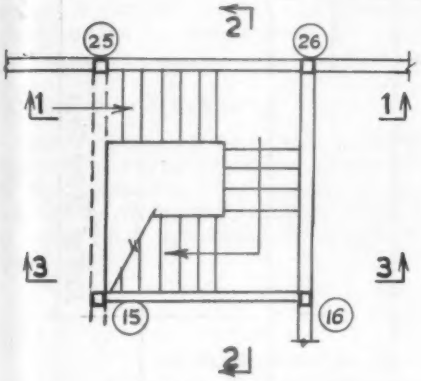


FIG. 5. FRAGMENT FROM A COLUMN SCHEDULE.

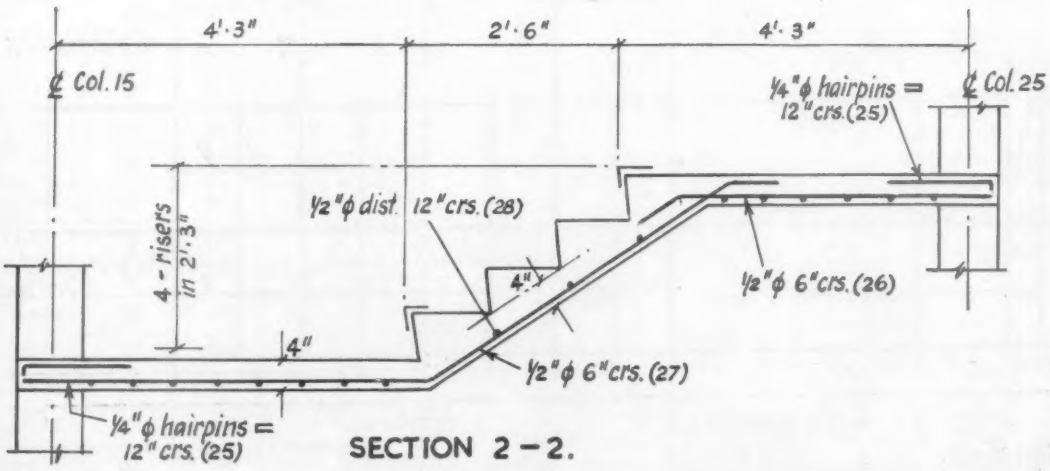


SECTION 1—1.



KEY PLAN

Section 3-3 omitted from this figure for convenience of illustration.



SECTION 2—2.

FIG. 6. DETAILING BY SECTIONS — STAIRS.

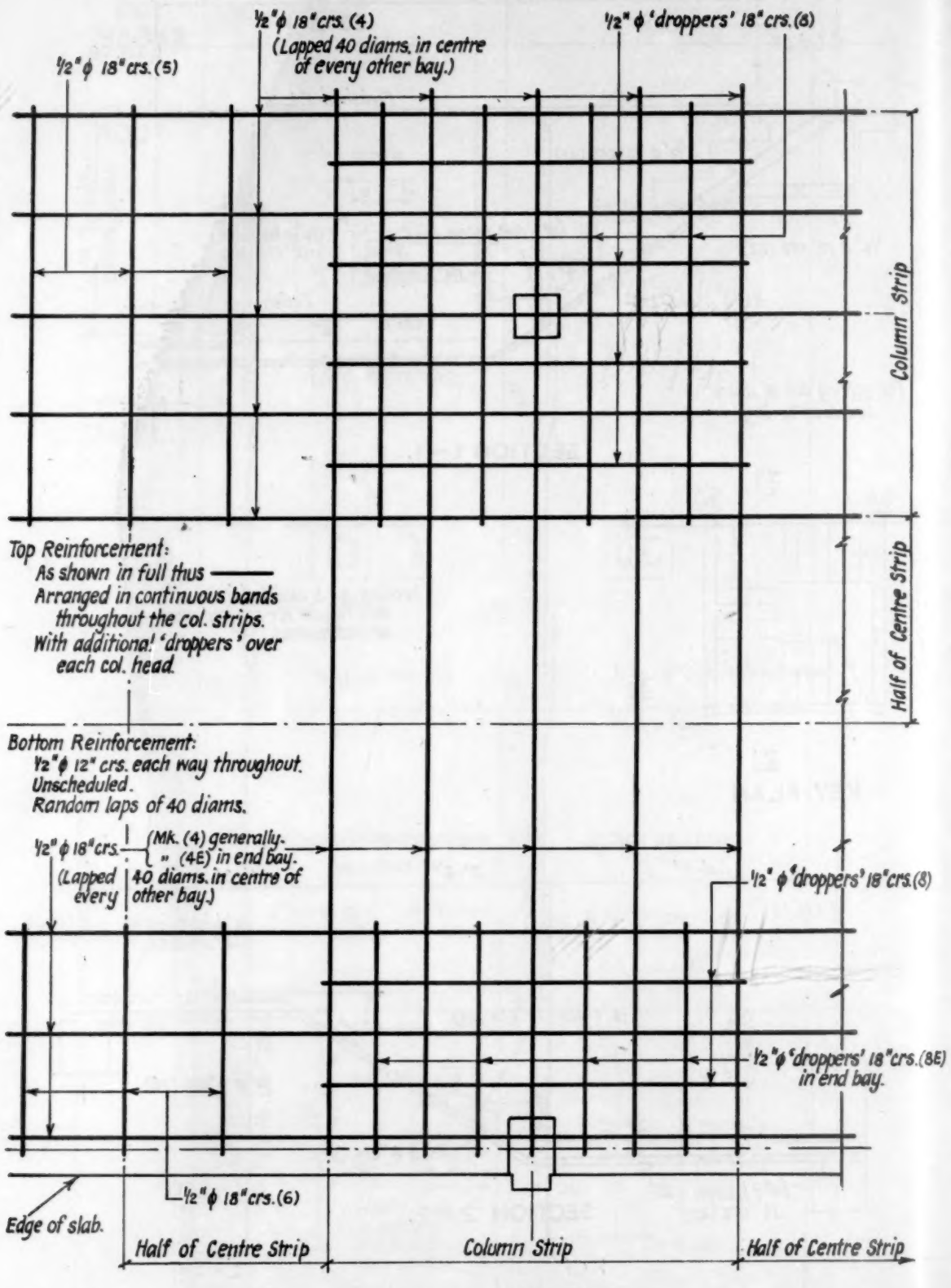

½"φ 18"crs. (4)
(Lapped 40 diams. in centre of every other bay.)
½"φ 'droppers' 18"crs. (8)
½"φ 18"crs. (5)
Column Strip
Half of Centre Strip
Top Reinforcement:
As shown in full thus ——
Arranged in continuous bands throughout the col. strips.
With additional 'droppers' over each col. head.
Bottom Reinforcement:
½"φ 12" crs. each way throughout.
Unscheduled.
Random laps of 40 diams.
½"φ 18"crs. (Lapped every
(Mk. (4) generally.
" (4E) in end bay.
40 diams. in centre of other bay.)
½"φ 'droppers' 18"crs. (8)
½"φ 'droppers' 18"crs. (8E) in end bay.
½"φ 18"crs. (6)
Edge of slab.
Half of Centre Strip
Column Strip
Half of Centre Strip

FIG. 7. FLUSH SLAB.

Large holes may be incorporated in a ' panel zone ' and detailed by sections. The important factor is to avoid showing endless reinforcement in plan.

In times of material shortages it is convenient to be able to replace one type of steel in short supply with another. This is most conveniently done by the straight substitution of alternative bars without adjustment to the centres of the reinforcement. In order to avoid difficulties with clearance the bar diameter is assumed, when detailing, to be $\frac{1}{8}$ inch larger than the nominal size.

The alternative sizes of steel reinforcement used in this way are shown in the Substitution Tables I and II. In these Tables it is assumed that compression bars in beams are tied-in by the reinforcement in the floor slab and compressive and tensile stresses are identical. Where this is not acceptable a suitable adjustment must be made.

Forms of Contract

Professional practice and Drawing Office procedure are closely associated with the contractual arrangements which are made for the construction of the work. There have been developments in recent years in current Forms of Contract and these are briefly reviewed in the following section.

(A) Fixed Price Contracts

Such a contract is made with a contractor for the construction of a particular project to a design prepared by the Consultant. The contractor quotes a lump sum for the work which must be fully defined by the drawings and specifications presented as tender documents.

Table I.
Substituted Reinforcement
Beams, Slabs, Walls and Foundations

Mild Steel Bar		Substituted Cold Worked Bar			
		Deformed		Square Twisted	
Dia.	Wt/Ft.	Dia.	Wt/Ft.	Square	Wt/Ft.
$\frac{1}{4}$″	·167	$\frac{1}{4}$″	·167	$\frac{1}{4}$″	·213
$\frac{5}{16}$″	·261	$\frac{5}{16}$″	·261	$\frac{1}{4}$″	·213
$\frac{3}{8}$″	·376	$\frac{5}{16}$″	·261	$\frac{5}{16}$″	·332
$\frac{1}{2}$″	·668	$\frac{7}{16}$″	·511	$\frac{3}{8}$″	·478
$\frac{5}{8}$″	1·043	$\frac{1}{2}$″	·668	$\frac{11}{16}$″	·700
$\frac{3}{4}$″	1·502	$\frac{5}{8}$″	1·043	$\frac{11}{16}$″	1·020
$\frac{7}{8}$″	2·044	$\frac{3}{4}$″	1·502	$\frac{5}{8}$″	1·330
1″	2·670	$\frac{7}{8}$″	2·044	$\frac{3}{4}$″	1·910
1$\frac{1}{4}$″	3·379	1″	2·670	$\frac{7}{8}$″	2·600
1$\frac{1}{4}$″	4·173	1$\frac{1}{8}$″	3·379	1″	3·400
1$\frac{3}{8}$″	5·049	1$\frac{1}{8}$″	3·379	1″	3·400
1$\frac{1}{2}$″	6·008	1$\frac{1}{4}$″	4·173	1$\frac{1}{8}$″	4·300
1$\frac{5}{8}$″	7·050	1$\frac{1}{4}$″	4·173	1$\frac{1}{8}$″	4·300
1$\frac{3}{4}$″	8·178	Not Rolled		1$\frac{1}{4}$″	5·320

Table II
Substituted Reinforcement
Columns

Mild Steel Bar		Substituted Cold Worked Bar			
		Deformed		Square Twisted	
Dia.	Wt/Ft.	Dia.	Wt/Ft.	Square	Wt/Ft.
$\frac{1}{4}$″	·668	$\frac{1}{4}$″	·668	$\frac{11}{16}$″	·700
$\frac{5}{8}$″	1·043	$\frac{5}{8}$″	1·043	$\frac{11}{16}$″	1·020
$\frac{3}{4}$″	1·502	$\frac{3}{4}$″	1·502	$\frac{5}{8}$″	1·330
$\frac{7}{8}$″	2·044	$\frac{7}{8}$″	2·044	$\frac{3}{4}$″	1·910
1″	2·670	1″	2·670	$\frac{7}{8}$″	2·600
1$\frac{1}{4}$″	3·379	1$\frac{1}{8}$″	3·379	1″	3·400
1$\frac{1}{4}$″	4·173	1$\frac{1}{8}$″	3·379	1″	3·400
1$\frac{3}{8}$″	5·049	1$\frac{1}{4}$″	4·173	1$\frac{1}{8}$″	4·300
1$\frac{1}{2}$″	6·008	Not Rolled		1$\frac{1}{4}$″	5·320
1$\frac{5}{8}$″	7·050			1$\frac{1}{4}$″	5·320

A Bill of Quantities is prepared of the materials and labours and priced by the contractor. The individual items may be used for adjustment to the lump sum tender for the work if this is modified or adjusted from that shown on the contract drawings.

The drawings required for contract are not generally sufficient for actual ' construction ' and are supplemented by additional information. This may be added to the contract drawings or prepared as supplementary details. These supplementary details may fill out the information shown on the contract drawings but they must not modify or extend the work.

In the case of a contract for the supply and erection of steel framework the supplementary details include the shop drawings required for fabrication of the material. These are generally prepared by the contractor to the approval of the Engineer. For reinforced concrete work the supplementary details will include the bar schedules and drawings of secondary items such as beam to column junctions.

As a variation, the reinforcement is included in the Bill of Quantities as a Schedule of Prices and details are subsequently prepared for construction. The concrete and formwork are fully measured in the Bill from the profile drawings. This modified procedure achieves a considerable economy in drawing office effort.

Fixed price contracts are generally made with a general contractor and include all specialist sub-contractors such as steel framework or reinforcement. The general contractor makes his own arrangements with sub-contractors for the specialist items.

In a fixed price contract the client gains the advantage of stability in prices. Variations in wages and the cost of materials which are made during the period of construction must be accepted by the contractor without passing on the charge.

In this type of contract the work is more fully detailed, generally, than in other forms. This does ensure that ' second thoughts ' and their consequent alterations are incorporated in the drawings and agreed with the contractor before the work begins at site. Some ' cost-advantage ' is generally achieved on this account.

The extra time required for drawing extends the interval before the contract is let. The fixed price

aspect will also tend to operate adversely in terms of speed when placed in competition for labour with contracts operating on a different basis. On these counts the project will generally take longer to complete.

(B) *Schedule of Prices*

For such a contract the Consultant's office prepares outline drawings indicating the basic design, tentative profiles and typical details together with a specification. A schedule of materials and labour is prepared from these which is priced by the contractor.

The priced Schedule becomes the basis for assessing the value of the work which is measured as executed on site. In practice there are many factors which tend to increase the cost of the work as compared with the fixed price method.

The type of working drawings which are finally required will be much the same as for a fixed price contract. An early start is made with actual construction at site but the drawings are generally prepared as the work proceeds.

The pressure from the construction programme will tend to reduce the time employed in drawing whilst the overall cost of the project will generally be greater. On both counts the actual cost of the drawings, expressed as a percentage of the overall cost of the work will be generally less than when the work is constructed by a fixed price contract.

(C) *Functional Specification*

In this type of contract the Consultant's office prepare an outline and specification of the basic requirements of the project. The contractor is required to submit construction proposals together with a lump sum tender which satisfies the functional conditions.

No structural design or proportions of the framework are required to be shown in the contract drawings. It may not be essential to include a structural drawing other than a layout of the site with subsoil conditions and overall dimensions of the project.

Such a method carries the disadvantage that a number of contractors are required to prepare embryo designs and prices for the same project. All the schemes submitted must be examined by the Engineer to enable a fair comparison to be made of the alternatives.

It might be noted that this is a much more difficult operation than appears at first sight. Codes of Practice for different forms of construction are most difficult to reconcile on strictly comparable standards of efficiency and amenity.

It is not generally suitable to employ this method of tendering for the 'framework' only. When comparing different forms of constructions, the 'framework' comprises different parts of the whole project depending on the extent to which composite action is invoked. Variation in the framework also modifies the cladding and finishes.

This type of contract allows an early start to be made on construction at site. It is also extremely difficult for the client or his representative to influence effectively the planning of the project as developed by the contractor without incurring expensive extras. In general, the finished structure cannot be tailor-made to the client's requirements but the standard achieved may, of course, be quite adequate for his purpose.

The time spent by the Engineer on such a project is low and the cost of the drawing office time spent by the builder is usually included in the overall cost of the work. The percentage cost to the Engineer is generally lowered on both counts.

This method is sometimes referred to as a 'package deal' in which the contractor supplies the building to his own specification. Too often, in such a case, the Engineer's services are dispensed with in the mistaken belief that his expert impartiality is unnecessary.

(D) *Target Scheme*

This type of contract is somewhat similar to that based on a Functional Specification. It avoids some of the disadvantages in the latter but is liable to others. The project is fully detailed in one form as for a fixed price contract. This chosen form of development is used as a 'Target Scheme.' Contractors are invited to quote for alternatives to the 'Target' which preserve the same main features.

This method of inviting tenders ensures that the chosen scheme will embody all the required features. It does impose some restriction on the flexibility of the alternatives especially where these have been developed to their own standard pattern. It will be found, generally, that a 'standard article' is more expensive than the 'tailor-made' scheme.

Each type of construction has its own most efficient form of development. The 'Target Scheme' has been planned to take advantage of this in its own medium. It is not generally economic to compress one form into the outlines developed for another.

In such a contract the work required of the Engineer's Drawing Office is similar to that required for a 'fixed price' contract. If the alternative is chosen, the Consultant is relieved from preparing supplementary details as required for construction. This saving is offset by the time spent in checking the contractor's scheme.

(E) *Development Teams*

This arrangement is sometimes referred to as 'parallel working.' The project is developed throughout by a team of specialists which includes the Architect, Consulting Engineer, Quantity Surveyor and the Contractor. The team is appointed at the outset and deals with all stages of development, from the initial sketch plans to completion of construction.

This procedure ensures that the design and construction are closely integrated. The success of the enterprise is dependent on the capacity of the group.

The major disadvantage of this method is the absence of the competitive element. Costs aspects are examined by the group as a whole and are a reflection of the efficiency of individual members of the group.

The design of the structural frame and working drawings are usually undertaken by the office of the Consultant. The supplementary details are developed by the Engineer and Contractor working in conjunction.

Conclusions

The following conclusions should be read in conjunction with the main text of which they form a very brief summarised version.

(1) The major structural decision to be taken on any new project is found in the choice of 'form and plan' having regard to the relative demands of 'cost,' 'amenity' and 'speed of construction.' This is best made as an independent appraisal of the many alternatives available.

(2) The structural design and working drawings are developed by highly specialised engineers, designers and draughtsmen. Such staff are a most valuable asset of the community and their efforts must be applied with the utmost efficiency.
(3) The cost of preparing the structural design and working drawings must be added to the cost of the work. The actual magnitude is approximately the same whether made as a separate charge by the specialist Engineer or absorbed in the cost of materials by the contractor.
(4) The conventional charges in this respect are calculated on the overall cost of the work and are required to cover staff salaries and overheads. The balance of fees and expenses in this computation is now finely poised and calls for the utmost economy in Drawing Office effort.
(5) The choice of 'form and plan' will be a predominant influence on overall costs of construction. Modification of the detail will have a relatively small effect in this respect.
(6) This encourages the use of repetition and simplicity of detail and leads to the first principle of conservation of effort, namely, "Reduce costs by the selection of 'form and layout' and use additional material to simplify the details."
(7) The efficiency of operations in the design and drawing office is improved and 'drudgery' is removed by planning these to a budget. The 'target-times' which may be allowed for the development of a particular project can be assessed by means of 'work-values.'
(8) In this respect it is valuable to employ simplified design procedures for general cases and to obtain assistance from standard computer programmes for complex framework.
(9) In the development of the project it is important to separate 'profiles' from 'working details.' The latter can be greatly simplified and clarified with 'diagrammatic' presentation.
(10) Forms of contract are now varied and complex. They affect the distribution of design and drawing effort required of the Engineer and contractor.

References

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Book Reviews

Highway Bridge Vibrations. I. Review of Previous Studies, by D. T. Wright and R. Green. (Canada: Queens University, Kingston, Ontario, Report No. 4, 1959). pp. 58.

This is a report of an investigation conducted in the Department of Civil Engineering of Queens University, Kingston, Ontario, in co-operation with the Ontario Department of Highways on the general topic of highway bridge vibrations and presents the results of a critical review of previous studies. The investigations reviewed and discussed include analytical, model and field studies of the action of moving loads on simple beams, highway bridges, and in certain cases, railway bridges and they are concerned with the prediction and measurements of impact, deflexion, stress and load distribution, and the vibration susceptibility of differing highway bridge types. The report is intended to provide an integrated description of the present state of knowledge of those factors influencing the dynamic deflexion of simple and continuous spans under the action of moving loads and for typical vehicles.

Technischer Strahlenschutz. I. Strahlenabschirmung durch Beton. II. Beiseitigung radioaktiver Abfallstoffe, by T. Jaeger. (Munich: Verlag Karl Thiemig, K.G., 1959). 8 in. × 5½ in., 192 pp., 86 figs., 13 tables. D.M. 8.60.

This booklet is made up of articles which were originally published in the journal "Atomkernenergie." The first part deals with radiation shielding of nuclear reactors and other such plant by high-density concrete, and the second part is concerned with methods for the disposal of radioactive waste. Most of the information has been obtained from research carried out in the United Kingdom and the U.S.A., and this has been acknowledged in the comprehensive bibliographies given at the end of each part of the book.

Welding Handbook, 4th Edition, Section III, Special Processes and Cutting, Edited by A. L. Phillips. (New York: American Welding Society, 1960; London: Cleaver-Hulme Press). 9 in. × 6 in., 500 pp. Illustrated, 72s.

This is the third of the five parts of the revised edition of the American Welding Society Handbook, one part of which is published each year. The material in Section III has been obtained from experts in each particular field irrespective of country. It contains a large amount of original material, and includes chapters on adhesive bonding and the welding of plastics. Other chapters deal with forge, thermit and induction welding, surfacing, metal and ceramic spraying, brazing, soldering, oxygen and arc cutting, ultrasonic welding, welding by cold working and stud welding. Each chapter concludes with a bibliography.

The volume contains 139 line and 196 half-tone illustrations and there are 78 tables.

Dock and Harbour Engineering, Vol. 3, Buildings and Equipment, by H. F. Cornick. (London: Griffin, 1960). 11 in. × 8½ in., 320 plus xvi pp., 126s.

This is the third of four volumes of an important new treatise based on the books "Dock Engineering" and "Harbour Engineering," by Brysson Cunningham, of which Volumes I and II were reviewed in THE STRUCTURAL ENGINEER in February, 1959 and March, 1960.

The chapters in this volume deal with dock roads and railways, floating and movable bridges, transit sheds, warehouses, grain silos and other buildings, working equipment, operating plant, bulk handling plant and auxiliary services and appliances.

An important feature of the volume is the description of existing and earlier installations, including numerous diagrams and photographs and each chapter concludes with a useful bibliography.

The Plastic Design of Latticed Portal Frames*

by

B. H. Fisher, B.Sc., A.M.I.Struct.E., A.M.I.C.E.,

Jacques Heyman, M.A., Ph.D., A.M.I.C.E.

and

L. G. Jaeger, M.A., Ph.D., A.M.I.C.E.

Introduction

The Paper describes the design and development of a range of prefabricated steel-framed single storey portal buildings. Rafters are of tubular lattice construction, while universal beam sections are used for the columns. Plastic theory is applied to obtain a load factor design of the portal frames.

The Paper is divided into four parts : the first describes the history of the project, and the second the theoretical design approach ; the third part describes a full-scale loading test carried out to destruction on a 50 ft. span portal building, whilst the fourth part discusses the test results and the conclusions to be drawn from the plastic analysis.

Part 1

History of Project

The design and development of the portal buildings was carried out by the first named author's firm acting as Consultants to a group of manufacturing companies. The collaboration between these Sponsor Companies and the Consultants has been continuous since the end of the 1939-1945 war and was formed for the purpose of carrying out, in collaboration, research and development work on the use of materials in the field of building and on techniques of building construction. The first results of this collaboration bore fruit in the design and fabrication of prefabricated houses of which a very large number were supplied by the Sponsor Companies.

The need for standard prefabricated buildings in the export markets of the world as well as at home was then further investigated, and as a result the design of a whole series of buildings was commissioned. The process of design was a gradual one, the range of buildings being revised and extended as markets developed. The buildings are all basically steel-framed, single-storey, using tubular lattice roof trusses, tubular purlins and mainly R.S.J. sheeting rails and columns. Table 1 shows the different series of standard buildings available at the end of 1960, and it will be noted that these cover a great variety of roof pitches, heights, spans and walling systems. This Table also illustrates the type of market for which the various buildings were designed.

Paper to be read before The Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 26th October, 1961, at 6 p.m.

In particular, it is to be noted that the storage buildings Mark I and II together with the new industrial buildings, which are intended to replace the former, have been designed for both the home and export markets. This meant that all structural members were designed for a superload of 15 lb together with a wind speed of 75 m.p.h. with extra bracing available for 100 m.p.h. wind conditions.

In 1959 it was decided to attempt to reduce the cost of standard buildings overseas by designing a new range for the export market only. The design specification for this range was as follows :—

(1) Superimposed loading.
This was reduced to 6lb. per sq. ft. as the buildings were intended for those parts of the world not subject to snow loading.

(2) Wind loading.
This was increased to 100 m.p.h. which was found to be a frequent requirement in overseas markets.

(3) Ease of assembly, recognition and erection.
It was most important that the assembly, recognition of components and erection should be as clear and simple as possible, since these buildings would in most cases be erected by unskilled labour and with the minimum of supervision.

The Consultants therefore proceeded to carry out design and cost analyses for approximately 40 different structural framing arrangements. Each structural arrangement was fully designed for a 50 ft. span building.

It was realised that eventually a whole range of spans would have to be designed, but as the 50 ft. span sold in the greatest quantities it was agreed that it would be safe to draw conclusions from designs based on this span.

The investigation covered the following variables :—

1. *Framing Systems* :
   (a) Simply supported trusses and cantilever columns.
   (b) Portal framing
2. *Bay and Purlin Spacings*
3. *Purlin Forms*
   (a) Tubes
   (b) Angles
   (c) Cold formed Z-sections
4. *Truss and Rafter Forms*
   (a) Hot Rolled Sections
   (b) Tubes

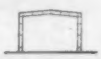
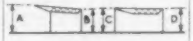
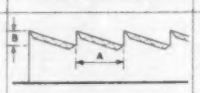
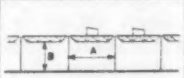
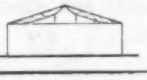
NAME	SECTION	SPANS	HEIGHTS	MARKET	
PORTABLE HUT		10'–3"	9'–0" TO EAVES	SMALL SPAN COVERED SPACE.	EXPORT.
TROPICAL ROOF		30'–0"	9'–0" 15'–0" TO EAVES	BUNGALOWS, LIGHT STORAGE OFFICES, SCHOOLS, COVERED MARKETS	EXPORT.
ARCON ROOF		21'–8" 26'–8" 35'–0"	9'–0" 12'–0" 14'–6"	SCHOOLS, HOSPITALS, BUNGALOWS, LIGHT STORAGE OFFICES.	EXPORT
THREE PIN FRAME		23'–0" WITH 7' & 11' EXTENSIONS	9'–6" A 9'–8 B 8'–2" C	SCHOOLS & COVERED MARKETS.	EXPORT
MONOPITCH		12'–2" 17'–2"	10'–11" A 9'–2" B 9'–4" C 9'–0" D	NATIVE HOUSING.	EXPORT
STORAGE BUILDING MK. I		33'–4" 40'–0" 50'–0"	10'–6" 14'–6" 20'–6" TO EAVES	STORAGE & INDUSTRIAL USES.	EXPORT & HOME
STORAGE BUILDING MK. II		58'–4" 66'–8" 75'–0"	10'–6" 14'–6" 20'–6"	STORAGE & INDUSTRIAL USES.	EXPORT & HOME
SAWTOOTH ROOF		50'–0" 16'–8" A	14'–6" 20'–0" 5'–0" B	INDUSTRIAL.	EXPORT & HOME
WIDE SPAN		100'–0"	14'–6" 20'–0" 27'–0" TO EAVES	INDUSTRIAL	EXPORT & HOME
MONITOR ROOF		30'–0" 40'–0" A	15'–0" B	INDUSTRIAL	HOME
MULTIFORM		20'–0" 24'–0"	10'–0" A	OFFICES, SCHOOLS, LABORATORIES HOSPITALS TECHNICAL COLLEGES	HOME
NEW INDUSTRIAL BUILDING. (to replace Storage Buildings Mk. I and II)		33'–4" 41'–8" 50'–0" 58'–4" 66'–8" 75'–0"	10'–6" 14'–6" 20'–6"	STORAGE & INDUSTRIAL USES.	EXPORT & HOME

Table 1.

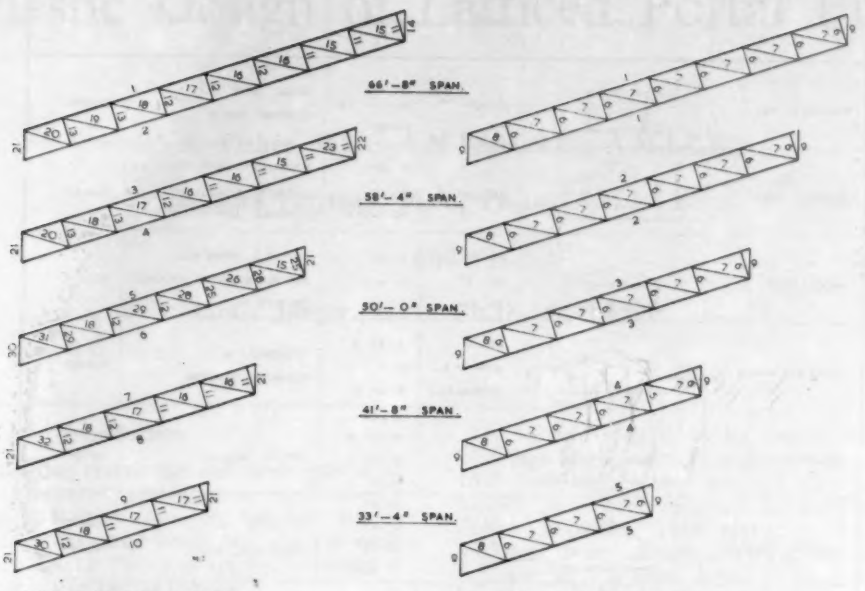
Brief details of standard buildings available

Components were designed, detailed and costs obtained from the manufacturers. In each case a building 100 ft. long x 50 ft. wide x 15 ft. high was priced, including the cost of roofing and sheeting materials. From this price comparison it was decided that the structural system which was the most economical and fulfilled the various marketing conditions complied with the following specification :—

1. Portal Type Framing.
2. 16 ft. 8 in. bay spacing and 4 ft. 4½ in. purlin spacing.
3. Purlins to be cold formed Z-purlins, each purlin being braced to the bottom boom of the portal rafter at each end.
4. Rafters to be of parallel boom tubular construction with columns of universal sections.

Once this decision had been taken, the Consultants proceeded with the structural design for the whole range of buildings. This involved the following different cases :—

(a) 5 No. spans 33 ft. 4 in., 41 ft. 8 in., 50 ft. 0 in., 58 ft. 4 in., and 66 ft. 8 in.
(b) 2 No. heights 15 ft. and 21 ft.
(c) 2 No. walling conditions.
    1. Sheeted with sheeting rail restraint to columns.
    2. Masonry walling without sheeting rail restraint to columns.
(d) 100 m.p.h. wind conditions.
(e) Double and single-span buildings for each of the above cases.



SCHEDULE OF TUBE COMPONENTS
ELASTIC ANALYSIS.

tube compon't.	size.
1	$3\frac{1}{2}$"Ø x 6g
2	$4\frac{1}{2}$ Ø x 7g
3	3" Ø x 7g
4	3" Ø x 5g
5	3" Ø x 10g
6	3" Ø x 7g
7	3" Ø x 10g
8	$2\frac{5}{8}$"Ø x 8g
9	$1\frac{5}{16}$"Ø x 8g
10 & 19	$1\frac{25}{32}$"Ø x 10g
11, 21, 23 & 31	$2\frac{3}{32}$" Ø x 10g
12 & 15	$1\frac{1}{16}$" Ø x 10g
13 14 16 & 32	$1\frac{9}{32}$" Ø
17 & 20	$1\frac{9}{32}$" Ø x 8g
18	$1\frac{11}{16}$" Ø x 10g
22	$1\frac{13}{32}$"Ø x 6g
24, 25 & 30	$1\frac{1}{16}$" Ø x 13g
26 & 29	$1\frac{15}{32}$" Ø x 12g
27 & 28	$1\frac{5}{16}$" Ø x 12g

SCHEDULE OF TUBE COMPONENTS
PLASTIC ANALYSIS.

tube compon't.	size.
1	$3\frac{1}{2}$"Ø x 6g
2	3" Ø x 7g
3	3" Ø x 10g
4	$2\frac{3}{8}$"Ø x 10g
5	$1\frac{13}{16}$"Ø x 8g
6	$1\frac{9}{32}$"Ø x 12g
7	$1\frac{5}{16}$" Ø x 10g
8	$1\frac{11}{32}$"Ø x 8g
9	$1\frac{1}{16}$" Ø x 10g

Table 2.

Comparison of rafter sizes designed by elastic and plastic analysis

These combinations necessitated the analysis and design of over 80 rigid frames. In the first instance the method of analysis was as follows :—

1. An idealised rafter was chosen and the horizontal and vertical reactions obtained by the " moment area method " of design.
2. These reactions were then used for constructing force diagrams from which the members of the rafter were sized.
3. The analyses were then checked on the computer at the Manchester School of Science and Technology by means of Dr. Livesley's partitioned plane frame programme.

In many cases, the desk work consisted of working out the sizes of the members for the vertical dead-plus-super loading conditions only, the check for wind conditions being carried out on the computer. It was possible to do this as in very few cases were the 100 m.p.h. wind conditions critical.

A further decision was taken which had a great influence on the final rafter components. In order to reduce the amount of jigging, stocking and the administrative problems of the manufacturers and marketing agents it was essential to have a single half rafter component for each span for both single and multispan buildings.

The design calculations, which up to this stage had been based on the elastic theory, showed that for a two-span building the portal rafters would be heavier than for single-span buildings due to the large negative moment over the central support. Hence the rafter component for each of the spans would have to be designed for the two-span conditions and would be uneconomic when used for single-span buildings. It was therefore decided to investigate whether the increase in rafter section for the two-span building was in fact necessary or arose from limitations in the elastic design. It is known that by using plastic methods of analysis in the design of solid web portal frames a saving in weight may be achieved compared with an elastic analysis. Therefore Professor Sir John Baker was asked to investigate the possibility of applying plastic methods to the analysis of latticed members. The minimum load factors were specified as 1·75 for dead-plus-super loading and 1·40 for wind loading. This analysis and a full-scale loading test

carried out on a 50 ft. span portal building are described in Parts II and III of this Paper. The results of the investigation may however be summed up as follows :—

1. Plastic methods may be used for the analysis of the latticed rafter.
2. The load factor for a two span building using the same rafter as that designed for a single span building is slightly greater for the two span case. It was therefore possible to design the rafters for all the single span buildings and use them economically for multi-span cases.
3. Further economies were achieved by load factor design by sizing all the lacing members so that the same sub-components could be used for all the five different spans. It was also possible to ensure that the top and bottom booms for each of the spans were identical. Table II compares the sizes of the members of the rafters for each of the five spans designed in the first instance by elastic theories with the final sizes adopted after redesign by plastic theory.

From an inspection of Table II it may be seen that load factor design enabled two-fold savings to be made. In the first instance there was a saving due to the reduction in tube sizes which was offset slightly by the fact that connecting plates at the ends of trusses had to be increased in size. The second saving arose from the fact that for the five half-rafters designed by the elastic theory 32 different tubular sub-components were required, while for the final designs adopted after analysis by plastic theory it was found that only 9 sub-components were required. It was this second fact which enabled the fabricators of these trusses to offer considerable cost reductions. The group of Companies which manufactures and markets the standard buildings of necessity hold large stocks of sub-components and trusses to enable them to offer the delivery time which is essential to market standard buildings. The reduction of sub-components from 32 to 9 was therefore a factor of major importance to the fabricators.

Part II

Plastic Analysis

Out of the range of buildings being considered it was decided to investigate two designs in detail, using the Plastic Theory. The designs chosen were the 50 ft. single span shown in Fig. 1 (a) and the corresponding double span frames. There are two aspects to the theoretical investigations :—

(a) to find the collapse load factor of the structure corresponding to formation of a collapse mechanism

(b) to investigate whether the collapse mechanism will ever in fact be achieved, or be forestalled by development of instability in some part of the structure.

The theoretical work thus fell into three parts, namely the mechanism calculations, the stability of the rafter, and the stability of the stanchion. Details of the analysis of the single span case are now given.

The Single Span Case

Details of the structure :—

Stanchions

8 in. × 5¼ in. × 17 lb. U.B. sections

Rafters

Booms 3 in. o.d. × 10g Tube top and bottom

Normal transversals 1 11/32 in. o.d. × 12g Tube

Inclined transversals 1 11/16 in. o.d. × 10g Tube.

Loading :—

Dead = 8 lb. per sq. ft.

Super = 6 lb. per sq. ft.

Wind = 72 m.p.h. in these calculations. The final design was checked for 100 m.p.h. wind speed.

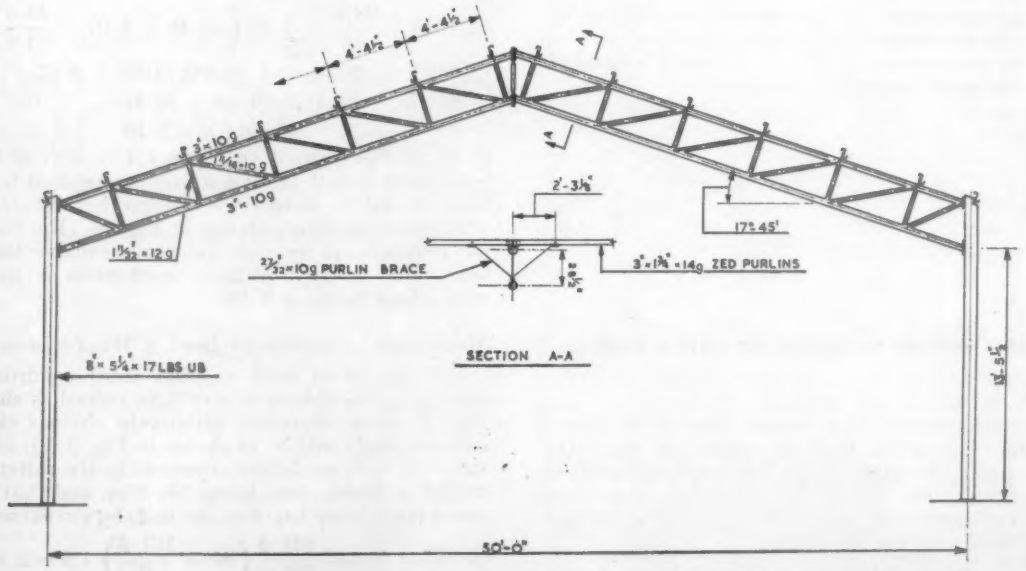


Fig. 1a.—Details of 50′ 0″ span portal frame.

Assumptions

The stanchion feet were taken to be pinned. The rafter was taken to be a pin jointed truss. Yield stress in the steel was taken as 16 tons per sq. in.

Mechanism Calculation for Vertical Loading (Dead + Super)

The plastic moment M_p of the stanchion = 253 tons-ins.
The collapse load of a boom member in tension (taking full yield value) is 18·4 tons. The collapse load of a boom member in compression was estimated by the Perry-Robertson formula to be 15·8 tons, using an effective length of 0.85 × panel length. Similarly for the transversals the collapse loads were estimated to be :—

Normal transversals :	6·5 tons tension
	5·2 tons compression
Inclined transversals :	10·0 tons tension
	5·9 tons compression

The technique used was to find a collapse mechanism in which collapse behaviour was exhibited in the stanchions and booms only, and then to check the transversals to verify that the permissible collapse loads were nowhere exceeded. For vertical loading the collapse mechanism is of the form shown in Fig. 1 (b) in which, for simplicity, the rafter is shown as a single member. Such a "line structure" is very useful (as will be seen later) for making very rapid approximate calculations on mechanisms and load factors. In the present case it is noted that, the collapse load of a boom member in compression being 15·8 tons and the booms being 28·5 in. apart, the plastic moment of the rafter in the line structure of Fig. 1 (b) should be taken as about 15·8 × 28·5 = 450 ton. ins. ; this is much larger than the plastic moment of the stanchion and so it is clear that the plastic hinge at the eaves will form in the stanchion. The mechanism of Fig. 1 (b) thus involves :—

(i) hinges at the pinned feet of the stanchions
(ii) plastic hinges in the stanchions at the level of the lower boom
(iii) plastic hinges in the rafters near the apex, this meaning in the actual structure a collapse in compression of one of the panel lengths of the upper boom near the apex.

Fig. 1b.—Collapse mechanism for vertical loading.

Proceeding directly to a consideration of the actual structure it is noted that by virtue of symmetry attention may be fixed on the left hand half only as shown in Fig. 1(c). In Fig. 1(c) λ (lambda) is the load factor at collapse. By symmetry the vertical reaction $V = \frac{1}{2}$ total load on the frame

$$= \left(\frac{50 \times 16{\cdot}67 \times 14}{2 \times 2240}\right)\lambda = 2.605\,\lambda \text{ tons.}$$

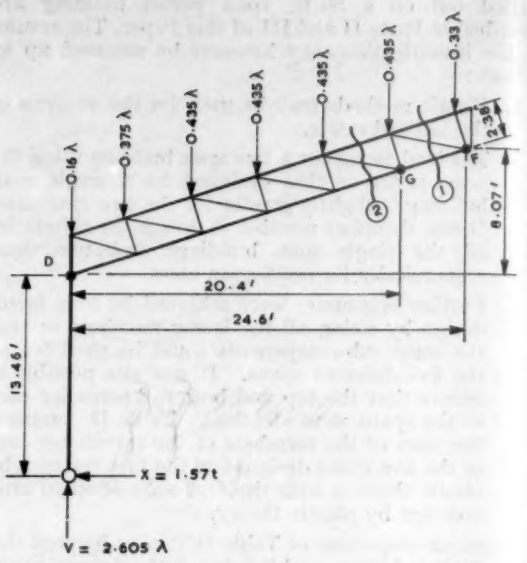


Fig. 1c.—Vertical loading in terms of λ the load factor.

Further for a plastic hinge in the stanchion at the point marked D the horizontal foot reaction is

$$x = \frac{\textit{stanchion plastic moment}}{\textit{height to lower boom}} = \frac{253}{161{\cdot}5} = 1{\cdot}57 \text{ ton}$$

It is assumed first that panel 1 is critical, with a collapse thrust of 15.8 tons in the upper boom. Then moments about F for the portion of the frame to the left of a section through panel 1, gives :—

$$15{\cdot}8 \times \frac{28{\cdot}5}{12} + 1{\cdot}57\left(13{\cdot}46 + 8{\cdot}07 \times \frac{24{\cdot}6}{25{\cdot}0}\right) =$$

$$(2{\cdot}605\,\lambda \times 24{\cdot}6) - \lambda\,[0{\cdot}435\,(4.90 + 9{\cdot}07 + 13{\cdot}23 + 17{\cdot}40) + (0{\cdot}375 \times 21{\cdot}55) + (0{\cdot}16 \times 24{\cdot}6)]$$

Whence $\lambda = 2{\cdot}18$

It is next assumed that panel 2 is critical and taking moments about G then gives :—

$$15{\cdot}8 \times \frac{28{\cdot}5}{12} + 1{\cdot}57\left(13{\cdot}46 + 8{\cdot}07 \times \frac{20{\cdot}4}{25{\cdot}0}\right) =$$

$$(2{\cdot}605\,\lambda \times 20{\cdot}4) - \lambda\,[0{\cdot}435\,(4{\cdot}90 + 9{\cdot}07 + 13{\cdot}23) + (0{\cdot}375 \times 17{\cdot}4) + (0{\cdot}16 \times 20{\cdot}4)]$$

Whence $\lambda = 2{\cdot}19$

It is thus postulated that panel 1 is critical with a load factor $\lambda = 2{\cdot}18$, and statics is now used to check that no rafter member is loaded beyond collapse. This gives the forces shown in Fig. 2. The forces in the members are nowhere above the collapse load and hence the assumed collapse mechanism is justified, with a load factor = 2·18.

Mechanism calculation for Dead + Wind loading

For the (dead load + wind load) condition the loading on the rafters is very light indeed as shown in Fig. 3. It is, therefore, intuitively obvious that the collapse mode will be as shown in Fig. 3 (a), i.e. pure sidesway with no failure anywhere in the rafter. The height to lower boom being 161·5 in. and that to the upper boom being 191·5 in. one finds by virtual work :—

$$(0{\cdot}395 + 0{\cdot}922)\frac{161{\cdot}5}{191{\cdot}5}\left(30 + \frac{161{\cdot}5}{2}\right)\lambda\,\theta = 2 \times 253\,\theta$$

Whence $\lambda = 4{\cdot}11$

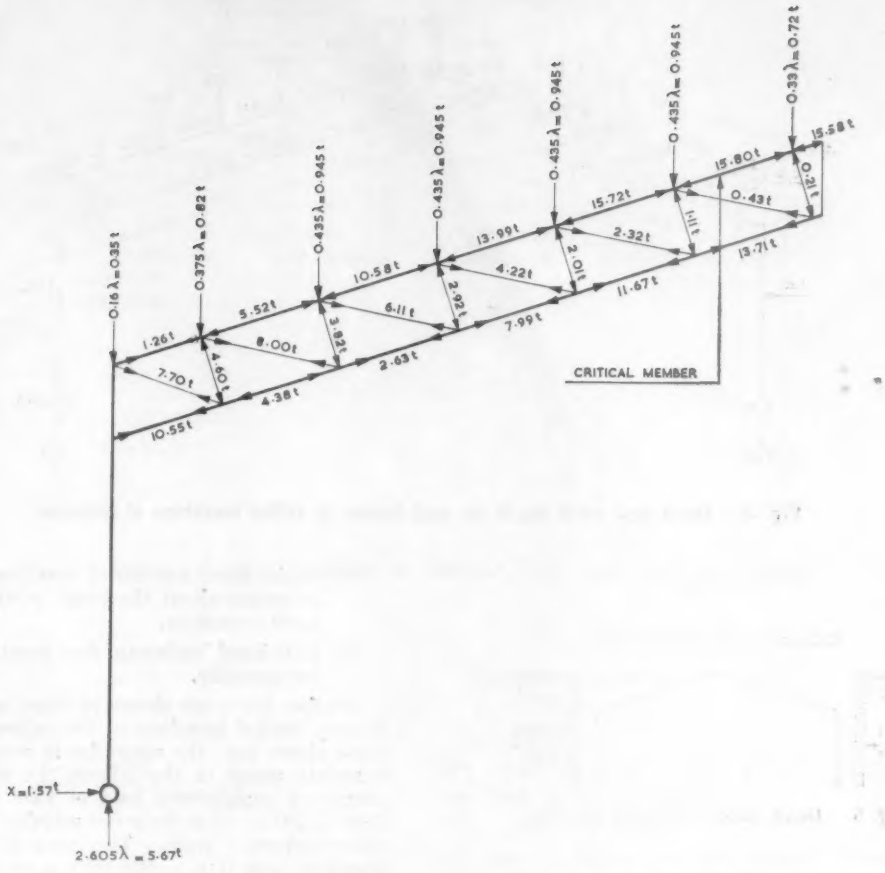


Fig. 2.—Forces in rafter members at collapse (dead plus super loading only).

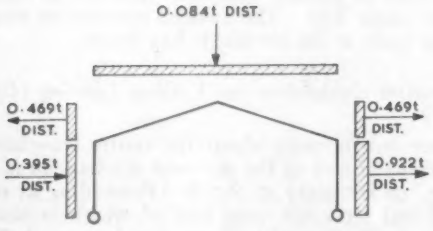


Fig. 3.—Dead and wind loading.

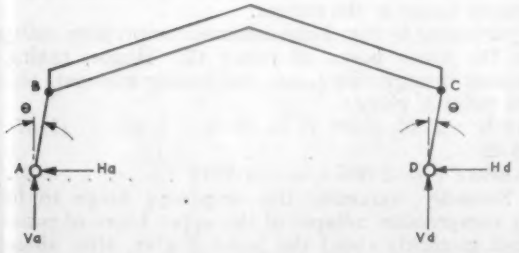


Fig. 3a.—Collapse mechanism for dead and wind loading.

The horizontal components of foot reaction are then found directly by taking moments about B (Fig. 3 (a)) for AB and about C for CD. They are $H_A = 2{\cdot}25$ tons and $H_D = 3{\cdot}16$ tons ; it is noted that these two check against the total horizontal load of 5·41 tons.

The vertical components of stanchion foot reaction are found by taking moments about D for the whole frame and about A for the whole frame respectively. They are $V_A = 0{\cdot}68$ tons (downwards) and $V_D = 1{\cdot}04$ tons (upwards) ; it is noted that these two check against the total collapse downwards load of 0·36 tons. Statics then gives the situation shown in Fig. 4. Not all the rafter member forces are shown ; they are all comfortably below collapse.

Mechanism Calculation for Dead + Super + Wind Loading

For the (dead + super + wind) condition the loads acting on the structure are shown in Fig. 5. Possible mechanisms are :

(a) Pure sidesway, for which the calculation given above for (dead + wind) load remains valid and so $\lambda = 4{\cdot}11$

(b) The mechanism shown in Fig. 6, in which the undisturbed position of the structure is shown in full lines and the disturbed position in dashed lines. I is the instantaneous centre for the right hand rafter.

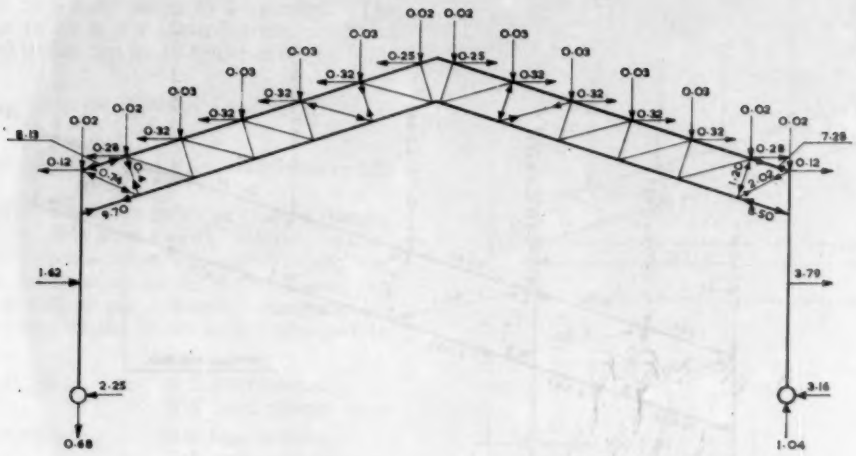


Fig. 4.—Dead and wind loads on and forces in rafter members at collapse.

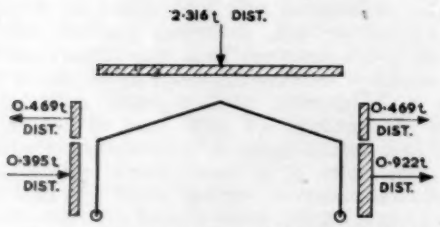


Fig. 5.—Dead, super and wind loading.

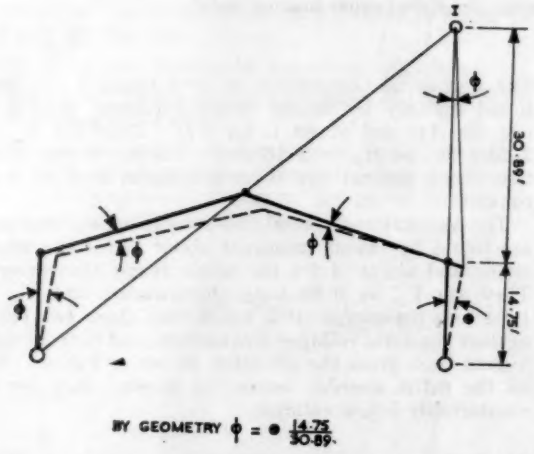


Fig. 6.—Collapse mechanism for dead, super and wind loading.

Applying virtual work to the line structure of Fig. 6 gives $\lambda = 2{\cdot}82$, which is much lower than the previous figure of 4.11. It is apparent therefore that a hinge will form in the rafter near the apex. Referring now to Fig. 7 and the actual structure, the reactions are found in terms of λ as follows :—

(i) Vertical foot reactions by moments for the whole frame about the stanchion feet.

(ii) Right hand horizontal foot reaction by taking moments about the hinge position in the right hand stanchion.

(iii) Left hand horizontal foot reaction by resolving horizontally.

Member forces are shown in terms of λ for the more heavily loaded members of the rafter. Inspection of these shows that the upper boom member in the first complete panel to the left of the apex reaches the permitted compressive load of 15·8 tons when $\lambda = 2{\cdot}88$ and that all of the other members are then within their permitted loads. This then is the load factor required, and it is noted that a very clear estimate (2·82) was obtained from the line structure of Fig. 6.

The Double Span Case

The mechanism calculations for the two bay frame are similar in general approach to those given above for the single bay. The various component members are the same as for the single bay frame.

Mechanism Calculation for Vertical Loading (Dead + Super)

There is symmetry about the centre stanchion and the left hand half of the assumed mechanism is shown in the line structure of Fig. 8. Proceeding at once to the actual structure, one half of which is shown in Fig. 9, the horizontal foot reaction is $x = 1{\cdot}57$ tons as for the single bay, this value being determined by the plastic hinge in the outer stanchion at lower boom level. This leaves the vertical foot reaction V and the load factor λ to be found by considering the plastic hinges in the rafters.

Referring to Fig. 9 one assumes compression collapse in the lower boom of panel 12. Hence, taking a section through this panel, and taking moments about the point K gives :—

$(15{\cdot}8 \times 2{\cdot}38) + 50\ V = (5{\cdot}21\ \lambda \times 25) + (1{\cdot}57 \times 15{\cdot}9)$

Whence $V - 2{\cdot}605\ \lambda = -0{\cdot}2514$ . . . . . .(1)

Secondly, assuming the remaining hinge to form by compression collapse of the upper boom of panel 1, then moments about the point F give, after simplification,

$V - 1{\cdot}277\ \lambda = 2{\cdot}891$ . . . . . .(2)

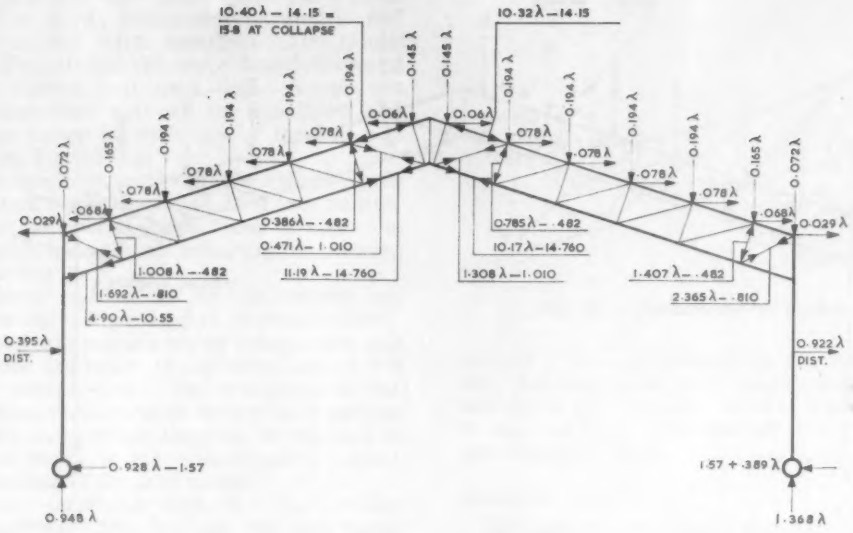


Fig. 7.—Forces in rafter members at collapse (dead, super and wind loading).

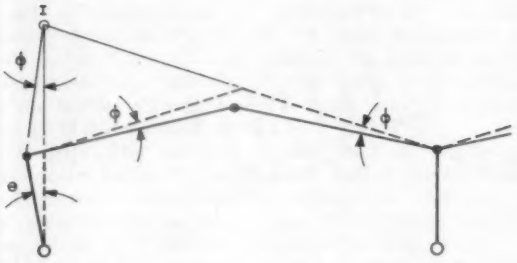


Fig. 8.—Collapse mechanism for vertical loading.

Equations (1) and (2) give :—

$V = 5 \cdot 91$ tons

$\lambda = 2 \cdot 366$

But if panel 2 rather than panel 1 is critical, then equation (2) is replaced by :—

$V - 1 \cdot 060\ \lambda = 3.382$ . . . . . .(3)

obtained by taking a section through 2 and moments about the point G.

Equations (1) and (3) give

$V = 5 \cdot 87$ tons

$\lambda = 2 \cdot 35$

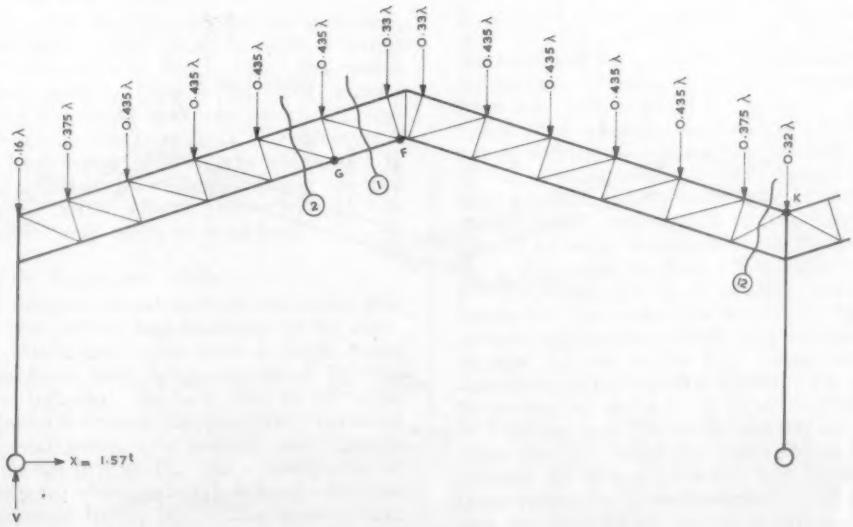


Fig. 9.—Vertical loading in terms of λ the load factor.

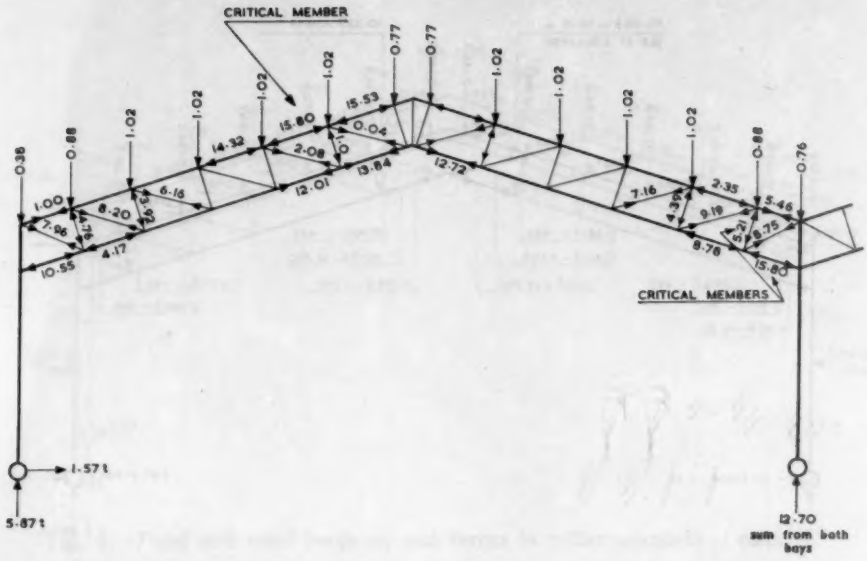


Fig. 10.—Forces in rafter members at collapse (dead plus super loading only).

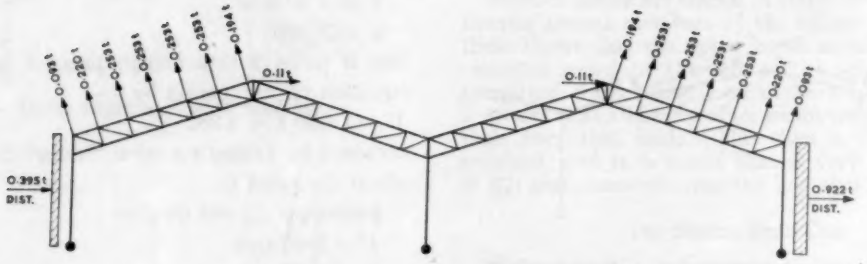


Fig. 11a.—Wind loading on two span frame.

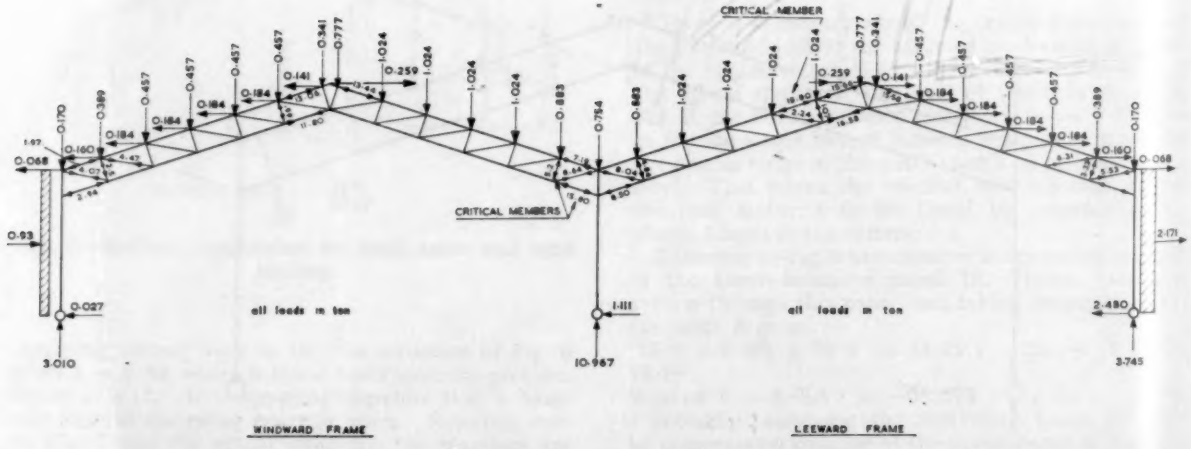


Fig. 11b.—Dead, super and wind loading on and forces in rafter members at collapse.

Hence panel 2 is just more critical than panel 1 and so one takes $\lambda = 2 \cdot 35$; statics may be used to find the forces in all the rafter members. The results are shown in Fig. 10 for the more heavily stressed regions of the rafters near their ends. Values not shown are intermediate between the maximum and minimum values shown for each type of member. It is then seen from Fig. 10 that :—

(i) All *boom* members are within the collapse loads of 15·8 tons compression and 18·4 tons tension, except at the critical panels 2 and 12 where the compression boom is just collapsing, as assumed in evaluating λ.

(ii) All inclined transversals are in tension and are below the collapse load of 10 tons (tension).

(iii) All normal transversals are in compression and all but one are below the collapse load of 5·2 tons in compression. The exception is the normal transversal next to the centre stanchion where the calculations show an excess load of 0·01 ton which is too insignificant to affect the estimation of the load factor.

It is therefore concluded that, for the two-bay frame with dead and super loading, the load factor $\lambda = 2 \cdot 35$.

Mechanism Calculation for Dead + Super + Wind Loading

For the two span case under (dead + super + wind) loading with 72 m.p.h. wind, there is added to the (dead + super) load of 14 lb. per sq. ft. the wind loading shown in Fig. 11 (a). A close estimate of the collapse load factor λ is quickly obtained using the " line structure " mentioned above and on proceeding to the actual structure the collapse condition shown in Fig. 11(b) is found, with $\lambda = 2 \cdot 355$.

Of particular interest is the way in which the mechanism forms in the left hand half of the structure. Two members must exhibit collapse in this part of the structure for a mechanism to be realised and the two which collapse are indicated in Fig. 11(b). It will be noticed that the result is not two " hinges " but one " hinge " and one deformable panel. This kind of behaviour has already been noticed (above) in the single span case, where one of the normal transversals went over its permitted load, though not by a sufficient amount to alter the load factor.

Also of interest is the fact that, for the two span case, the collapse load factor under (dead + super + wind) loading ($\lambda = 2 \cdot 355$) is almost identical with that under (dead + super) loading ($\lambda = 2 \cdot 35$). The virtual independence of λ from wind load was quickly checked by applying 100 m.p.h. conditions to a " line structure " and a collapse load factor of 2·36 was obtained. It happens that the relief of vertical loading on the outer parts of the rafter offsets almost exactly the effects of the other components of wind load.

The Stability of the Rafter as a whole

In the above mechanism calculations the rafter was assumed to be pin jointed and buckling of the compression chord was taken to be over a single panel spacing, the buckling load being estimated by the Perry-Robertson formula. In fact, due to the continuity of the boom members the possibility exists of a buckling mode extending over several panel spaces in the manner shown in Fig. 12. An investigation of rafter stability was therefore undertaken using a method proposed by Professor Horne (1).* This showed that the lowest value for buckling the compression boom

Figures in parenthesis refer to items in the bibliography.

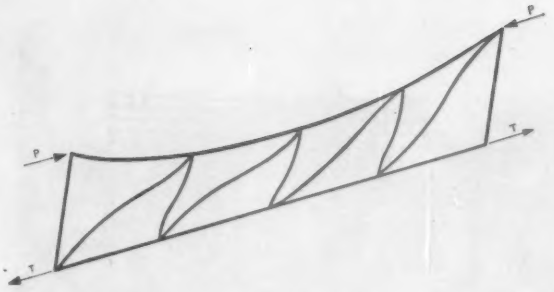


Fig. 12.—Possible rafter buckling mode.

was 25·7 tons, corresponding to development of a half sine wave over three panel spaces. This load is well above the " squash " load of a boom member and it was concluded that the structure was safe from this instability mode.

Stanchion Stability

The most severe case is that for stanchions which have no intermediate lateral support from sheeting rails, between lower boom level and the feet of the stanchions. First, it is of interest to note that on an elastic design basis, the stability is borderline in that it depends on the choice, which cannot be precise, of effective length. Thus, under dead plus super-loading, the outer stanchion just satisfies B.S.S. 449 if the effective length is about $0 \cdot 8L$ where $L = 161 \cdot 5$ in. and is the unsupported length between lower boom level and the foot.

For plastic design, the operative question is whether the fully-plastic moment can be developed at lower boom level, as assumed in the crucial collapse mechanism, before lateral instability of the stanchion occurs. In this connexion, use can be made of the results of Horne (2) who analyzed the problem of a strut of length l, with ends pinned about both flexural axes but fully restrained against twisting, loaded by a mean axial stress p together with major-axis bending moments of magnitude M_p at one end and βM_p at the other end. Horne's results are presented in the form of curves from which the critical slenderness ratio l/r_y can be read off for a given set of values of p, β and a section constant T; this last is defined by $T = AGK/Z^2_x$ where A is area, Z_x is major section modulus and GK is the St. Venant torsional rigidity of the section. Values of T for universal beam sections have been published (3).

For the present application, Professor Horne's curves for the case $\beta = 0$ are relevant, corresponding to free rotation of the stanchion foot about the major axis, and the appropriate value of p is 1·17 tons per sq. in. which corresponds to the calculated collapse load in an outer stanchion of the two bay frame under dead plus super-loading. The slightly smaller value $p = 1 \cdot 13$ tons per sq. in. is relevant to the single bay frame but this reduction is scarcely significant in the present application, which may be taken as applying equally to the single bay frame and to the outer stanchions of the two bay frame. The relevant section properties of an 8 in. × 5¼ in. × 17 lb. *U.B.* are $r_y = 1 \cdot 16$ ins. and $T = 20 \cdot 1$ tons per sq. in. This latter value lies just below the lower end of the values of T covered by Horne's curves, but linear exterpolation from values for $T = 24$ and $T = 30$ can be used to give the stability criterion $l \not> 118$ ins.; this compares with the unsupported length of 161·5 inches from foot to lower boom level.

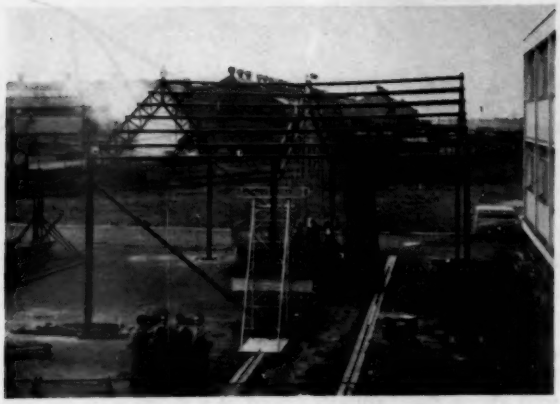


Fig. 13.—Test arrangement.

The stanchions clearly fail to satisfy a direct application of Horne's stability criterion. However, there are several differences between the actual stanchion conditions of the present problem and Horne's assumption which may have a significant effect on stability. Firstly, the stanchion foot is partially, if not completely, restrained against rotation about the minor axis. This restraint is beneficial and it is estimated that it might give an increase in critical length of order 10-15% in the present problem. Secondly, Horne's analysis errs on the side of safety in neglecting the beneficial contribution of warping rigidity to torsional stiffness. This contribution becomes relatively much more important for universal sections such as the 8 in. × 5¼ in. × 17 lb. *U.B.* with thin flanges and web and a consequently low St. Venant torsional rigidity. This contribution, in the present application, might well lead to an increase of 20% in the effective length. Thus the combined influence of warping rigidity and foot restraint could conceivably raise the critical length from $l = 118$ inches to about the value $l = 161 \cdot 5$ ins., corresponding to borderline stability. But a third factor must be mentioned which acts in the reverse direction (i.e. of tending to decrease stability). This factor is that the actual stanchion may not satisfy the particular assumption of Horne's analysis that the strut is completely restrained against twisting rotation at the plastic hinge, i.e. at lower boom level in the present problem. Some restraint will certainly be provided by the flexural stiffness of the lower boom and the braces from the purlins but no reliable estimate can be given of the magnitude and effect of any lack of complete torsional fixity of the stanchion at lower boom level. In view of this uncertainty, in particular, and of the sparseness of experimental data relevant to the problem of stability in the presence of plastic hinges, it was decided to carry out a full-scale test as described in the next section of the paper.

Part III

Full-Scale Test

A standard 50 ft. span portal, as analysed in Part II, was tested to destruction to observe the collapse load, and hence collapse load factor, as well as to confirm the predicted mode of collapse. The test arrangement, see the photograph Figure 13, consisted of loads applied to the centre portal only of three parallel frames. These three frames were erected in the normal way, complete with purlins, and wind bracing, but the purlin stays were omitted in the two

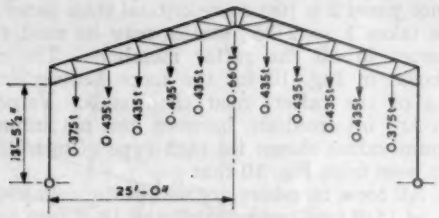


Fig. 14.—Details of test frames and loads.

gable frames to reduce load transference to the outer frames.

Increasing dead loads were applied to the portal under test through yokes at each of the lower-boom panel points; Figure 14 shows the leading dimensions of the frame and the calculated *working* loads at each of these panel points. Each loading cradle was made up to a starting weight of 0·05 tons (this refers to the eight equal loading points; the centre load and two outermost loads were increased and decreased respectively in proportion). Increments of load were then by 0·10 tons up to a total on each cradle of 0·35 tons, by 0·05 tons up to 0·65 tons, and thereafter by 0·025 tons until collapse occurred at 0·925 tons. The collapse load factor was thus 2·12.

Deflexion readings on the portal under test were made at the apex and at each mid-rafter length. Squared paper was attached to the portal at each of these points, and the movement observed relative to the fixed lines of sight of a theodolite and a number of levels. Apex deflexions were also recorded for the two outer frames, in order to estimate any possible transfer of load from the frame under test. In addition, stanchion deflexion readings were taken at lower boom level for the test frame. Three wires were attached to a horizontal angle fixed to the frame perpendicular to its plane. These wires passed over pulleys and hung vertically under the weight of bobs. Attached to each wire was a graticule whose movement could be read against squared paper. The central wire, in the plane of the frame, recorded directly the eaves movement of the test frame, and the difference between the readings of the two outer wires gave a measure of the twist of the stanchion at lower boom level. A schematic diagram showing positions of deflexion measurements on the test frame is given in Figure 15.

The whole frame was coated in plumbers' resin in order to observe visually the onset of yield.

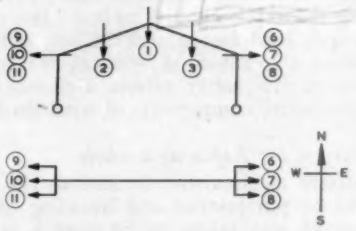


Fig. 15.—Diagram showing positions of deflexion measurements.

Main Test

The deflexion readings on the two outer frames indicated that, within the experimental error, no load was transferred to them from the frame under test.

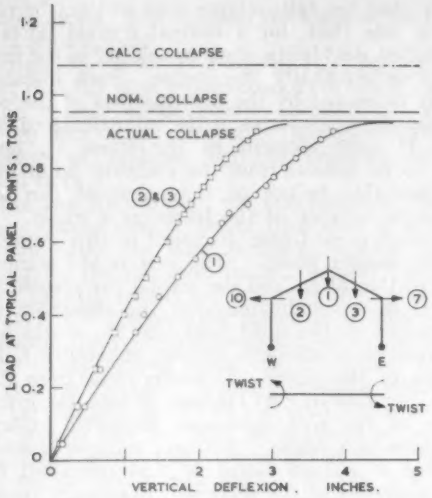


Fig. 16.—Vertical deflexion at apex and mid rafter point.

Load deflexion curves are given in Figures 16, 17 and 18. Figure 16 shows the vertical deflexions at apex and mid-rafter section, Figure 17 the eaves deflexions, and Figure 18 the twist of the stanchions at lower boom level. It will be seen that collapse occurred with the east stanchion forming part of the collapse mechanism, accompanied by considerable twisting at the top of the stanchion. The west stanchion also twisted slightly, but did not suffer noticeable plastic deformation, and the collapse mechanism is sketched in Figure 19. Final collapse of the portal was characterized by uncontrolled twisting at the top of the east stanchion, by increasing eaves deflexion at the same stanchion, and finally by plastic buckling of the top boom member over the first and perhaps the second panel nearest the apex in the west rafter, the buckling in the apex panel being much the more severe. Photographs showing the failures are given in Figures 20 and 21.

Cracking of the resin was first noticed at a load of about 0·5 tons, and occurred at the top of the east stanchion at the section of maximum subsequent

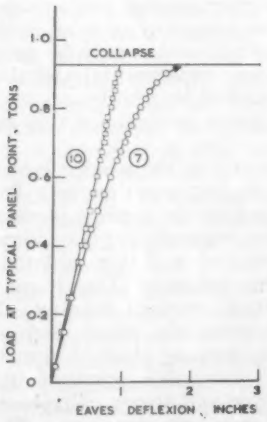


Fig. 17.—Eaves deflexion.

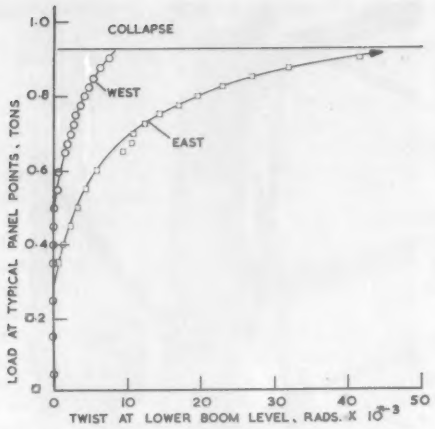


Fig. 18.—Twisting of stanchions at lower boom level.

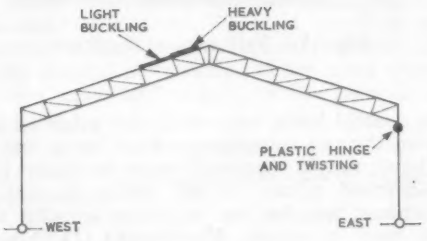


Fig. 19.—Collapse mechanism of test frame.

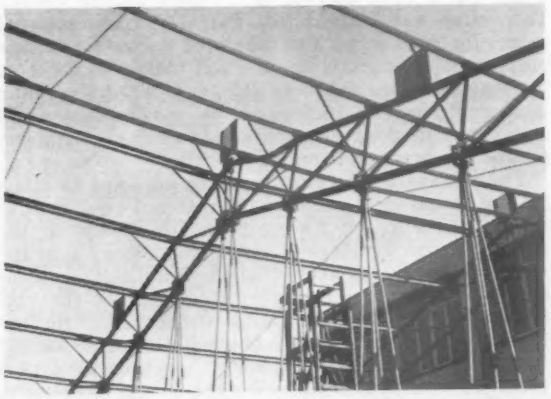


Fig. 20.—Failure in top boom of rafter.

twist. This cracking spread with each successive increment of load, but no cracks were observed elsewhere until final collapse of the portal.

Control Tests

Tensile tests were made on material cut from the tubular members of the rafters. The mean value of yield stress from the four tests was 18·7 tons per sq. in., with an expected statistical error of about ± 0·3 tons per sq. in.

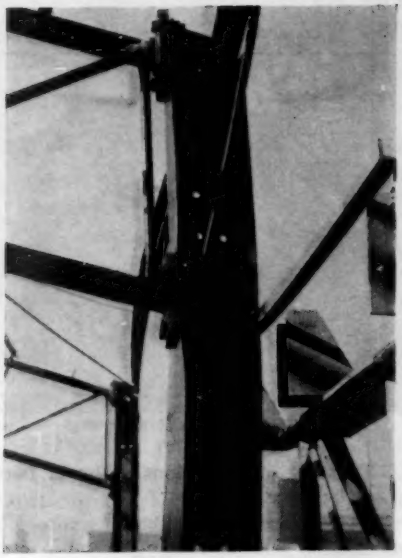


Fig. 21.—Failure of east column.

Two control beam tests were also made on lengths cut from the two stanchions. Each beam was tested to collapse, simply supported under two-point loading. The observed values of full plastic moment were 297 ton/ins. for the east stanchion and 321 ton/ins. for the west stanchion. This spread (7½%) is rather large, and the two stanchions may have come from different rollings.

Test Results

The nominal calculated collapse load factor for the frame was 2·18 (0·950 ton on a typical loading point). This value was derived (see Part II), using nominal values for yield stress and tabulated values for section properties. For example, the full plastic moment of the stanchion was taken as 253 ton/ins. corresponding to a yield stress of 16 tons per sq. in.; the same yield stress was used in determining the failure condition for the compression boom of the rafter.

The nominal load factor must be adjusted to allow for the actual yield stress of the material used, that of the boom being 18·7 tons per sq. in. Since in the actual test no significant yielding was observed in the west stanchion, it is likely that the full plastic moment of that stanchion was higher than that of the east stanchion. This conclusion is strengthened by the results of the two beam tests, and the figure of 297 ton/ins. for full plastic moment, together with 18·7 tons per sq. in. yield stress for the booms, is used to give a calculated load factor of 2·54. This figure compares with a value 2·12 observed. Fuller discussion of these test results is given in the next Part of this Paper.

Part IV

Discussion

In applying plastic theory to the design of steel structures, it must be remembered that that theory is based on certain assumptions. One of these is that the structure as a whole, and individual members, do not fail primarily by instability. Should stability be an important feature in the behaviour of any frame, then there is a danger, to a greater or lesser extent, that the full collapse load will not be reached.

It is true that, for a normal framed portal built from rolled steel joists, some instability of the members always occurs finally at collapse. Such instability is usually promoted by the deformations of the collapse mechanism, which forms at the full calculated collapse load. If some element in the frame is, however, liable to be critical from the stability point of view, then unstable failure of that element can lead to premature collapse of the frame as a whole.

In the type of frame discussed in this Paper, there are two danger points. The first is the compression boom of the rafter, and the second the stanchion. In Part II it was indicated that the stanchion section chosen was on the limit, and that there was little, if any, reserve of strength against instability failure at collapse of the frame. It seems clear from the test results reported in Part III that, in fact, the instability failure of the east stanchion promoted collapse of the frame as a whole. Whereas collapse should have occurred at a load factor of 2.54 (adjusted for the true properties of the sections), it actually took place at a factor of 2·12, a shortfall of some 16%. Had the material been exactly on the limit of the British Standards, then the nominal load factor of 2·18 would not have been realized, but a factor of about 1·85 would have been observed in the test.

The test revealed that the potential instability of the compression boom was not an important feature of the design. The final instability of the boom was promoted by the collapse of the frame as a whole, and not vice versa. The shortfall in collapse load factor is attributable, then, almost certainly to the behaviour of the east stanchion. The collapse load factor observed, even when adjusted to allow for the stronger material used, is above the nominal design factor of 1·75.

Despite the clear indications of promotion of collapse by instability of the east stanchion, it should be noted that the west stanchion remained stable. From the beam tests, the west stanchion was apparently about 8% stronger than the east; this small difference might completely account for the difference between stable and unstable behaviour. Alternatively, the initial imperfections of the east stanchion, or the possible small eccentricity of load transferred from the lower boom, may have been greater than for the west stanchion. In any case, it was clear from the test that twisting occurred at the outer flange of the stanchion at lower boom level, and complete fixity over the whole section was not obtained. Thus the condition of complete restraint against torsion was not observed.

However, the stanchion analysis used leaves something to be desired, and it may be mentioned that a new investigation, experimental and theoretical, is in progress at Cambridge in order to try to produce a more refined design method for this important class of stanchion.

The analysis of the compression boom is also somewhat suspect, relying as it does on a Perry-Robertson type formula and on the concept of effective length.

Despite these reservations, the collapse mechanism observed in the test is that postulated in the calculations, with pure sidesway of the frame superimposed. With symmetrical vertical loading, the theoretical collapse mechanism has extra degrees of freedom, both stanchions forming plastic hinges at lower boom level. Due to inevitable practical imperfections, a frame of this type will always collapse with a mechanism involving only one stanchion. The symmetrical and asymmetrical mechanisms form at the same load factor, and are in fact the same mechanism in two

alternative forms ; in both cases the loads in the frame members are identical.

Reviewing the work as a whole, it is immediately apparent that the plastic calculations are much simpler than the corresponding elastic calculations. As is usual, the fact that the frame becomes statically determinate at collapse lightens considerably the analytical work.

For the two span frame, the conclusion is reached that the collapse load is higher than that of a single span frame using the same sections. Elastic theory would require the two span frame, to carry the same loads, to have increased sections near the valley. It is, however, the compression members in this region which elastic theory indicates should be strengthened. Thus the valley section would also be critical for the plastic collapse mechanism, and premature instability failure here might lower the overall collapse load factor. It is of interest that, using the same sections as for a single span frame, and assuming the compression boom to be completely "knocked out" by instability at the valley, the two span frame could still carry load above the nominal load factor of 1·75 before collapsing. Thus, even if premature collapse of the boom occurred, and this is unlikely if its behaviour followed the general pattern of that in the single-span test, overall collapse of a two bay frame would not thereby be promoted.

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United Steel Companies Limited.

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It would be invidious to single out any particular member of the Architectural or Engineering Section of the firm for special mention, as so many assisted in carrying out the work described in the paper and helped in checking the text.

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Book Reviews

General Dynamics of Vibrations, by Y. Rocard. (London : Crosby Lockwood, 1960). 8½ in. × 5½ in. 522 + xii pp. 80s.

This book on the general dynamics of vibration is translated from the third Paris edition of "Dynamique Générale des Vibrations." In it the author has endeavoured to provide a course of higher physics relating specifically to vibrations, covering a wider field than most other books on the subject.

The volume is divided into two parts, the first dealing with vibrating systems and the second with accoustic waves. Part I has chapters on oscillatory systems, damping, the general concept of impedance and its introduction into various systems, gyroscopic coupling, electromechanical vibrating systems, shock excitation, filters, unstable systems, self-excited oscillations due to rolling constraints, problems of directional stability of vehicles, and studies of non-linear systems. The second part, entitled, accoustic waves, deals with radiation and diffraction, Huygens principle, studies of horns and trumpets and practical realisations.

Appendixes on oscillations depending on Mathieu's equation, torsional oscillations and oscillations due to "hunting" of sychronous motors and rotary convectors complete the volume.

Plasticity : Proceedings of the Second Symposium on Naval Structural Mechanics, held at Brown University, Rhode Island, 1960. Edited by E. H. Lee and P. S. Symonds. (Oxford : Pergamon Press, 1960). 10 in. × 6 in., 611 plus xviii pp., 70s.

This Symposium, devoted to plasticity, was sponsored jointly by the Office of Naval Research of the U.S. Navy and Brown University. The intention was to provide critical reviews of recent developments and to give information on current research work.

Leading experts from the United States, the United Kingdom and the Soviet Union took part in the symposium, and the papers delivered dealt with basic theory, basic experimental research and applications in engineering design. The six sessions dealt with the following subjects :—Problems of plasticity in naval structures and atomic theory of plasticity and fracture of crystalline solids, stress strain relations and thermo-plasticity, topics in the theory of plasticity, solutions of boundary value problems, plates and shells, dynamic loading and plastic waves and recent developments in design.

The volume is well-produced and well illustrated and each paper concludes with a comprehensive bibliography.

Some Notes on the Fatigue Resistance of Corroded Prestressing Wire

A. REPORT ON CERTAIN TESTS ON WIRE EXPOSED IN LAGOS

by

A. Goldstein, B.Sc.(Eng.)., M.I.Struct.E., M.I.C.E., M.Cons.E.*

NOTES ON SOME PUSH-PULL ALTERNATING STRESS TESTS ON CORRODED COLD DRAWN STRESS RELIEVED WIRE FOR PRESTRESSED CONCRETE

by

C. F. Brereton, F.I.M.†

PAPER A.

REPORT ON CERTAIN TESTS ON WIRE EXPOSED IN LAGOS.

Introduction

(1) From 1956 to 1958 during the construction of certain prestressed concrete works in Lagos, it was noted from inspection of wire deliveries that a significant proportion of wire exhibited varying degrees of corrosion and—in particular—pitting. Various tests were carried out on samples of this wire prior to the tests described in this paper. The tests, comprising physical and chemical examination, static tensile strength tests and bend tests, were carried out both by a number of wire manufacturers and an independent tester. In due course the wire showing significant pitting (this generally being coupled with corrosion) was replaced by wire free from these characteristics. Subsequently, however, the matter was examined further and particular attention was directed to the effects of pitting on fatigue resistance as measured by ' push-pull ' tests in an Amsler vibrophore. The results of these tests are reported below. Following a study of them, it was considered that further tests should be implemented on specimens which had corroded under controlled conditions with a view to ascertaining more general data than could be obtained from the necessarily *ad hoc* experiments on the samples of wire taken from Lagos. The results of those tests are reported separately in Paper B.

The present paper is divided into two sections being (a) summary of the data obtained on the wire prior to the study of fatigue resistance and (b) the results of the fatigue resistance experiments on samples of corroded wire obtained from Lagos.

* Partner, R. Travers Morgan and Partners.

† Chief Metallurgist—Messrs. Richard Johnson & Nephew Ltd., Manchester, to whom his thanks are due for permission to publish this note.

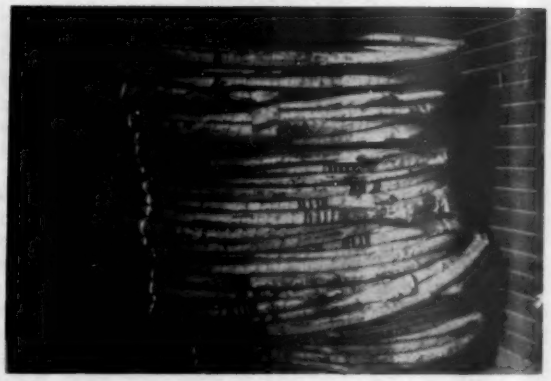

Fig. 1. (Paper A.)

Data on Wire prior to study of Fatigue Resistance

(2) The wire was the normal 0·276 in. diameter hard drawn (and in certain cases patented) wire delivered from the United Kingdom to Lagos by sea. It was manufactured by a number of well-known United Kingdom wire makers and was intended for use with the Freyssinet prestressing system. The wire was made up into 12-wire cables in the usual manner prior to shipping. These cables were then coiled in about 8 ft. coils which were wrapped with hessian based bitumen paper. They were taken in ships' holds to Lagos where they were unloaded and stored in coils under cover on site. The wire was delivered fairly early in the Contract (as is usual in countries lacking their own manufacturing facilities) and, consequently, there was a fairly lengthy storage period. A typical stack of wire thus stored is shown in Fig. 1. Some time well before the incorporation of the cables in the works they were taken from store and laid out straight under

cover (roof only, no sides or ends) so as to be ready for sheathing, ancillary operations and incorporation in the works.

(3) During this time it was noted that certain wires exhibited not only the normal degree of rusting which is associated with prestressing wire and which indeed is often thought preferable to bright wire for reasons of improved bond, but also a considerable amount of pitting in various places along the length of wires. This pitting varied from very light 'marking' visible only on most detailed inspection, to heavy 'cratering' which was readily visible. In certain cases the wire was free of corrosion pits on a surface band running the length of each wire and a little less than half the circumference in width, the remaining half exhibiting pitting.

(4) The Specification for the wire was for normal hard-drawn wire in accordance with B.S.2691 having a minimum ultimate tensile strength of 95 tons per sq. in. and 0·2% proof stress of not less than 85% of the ultimate strength. Clause 3 of B.S. 2691 states that wire shall be 'sound and free from splits, surface flaws, rough jagged and imperfect edges and all other defects.'

(5) Due to the method of manufacturing cables prior to shipping, the source of any particular wire could not be readily identified. Similarly, a great many of the delivery slips connected to coiled and wrapped cables were found to be missing when these cables were stored. A number of samples of wire exhibiting various degrees of corrosion pitting were sent to the several manufacturers and to an independent tester. The facts reported from an examination of these samples were as follows :—

(a) Using a point micrometer, the measured 'diameters' were 0·278 in. (in the best parts) and 0·263 in. (in the worst parts). The figure of 0·263 in. was a measure of local reduction in diameter, i.e., at a corrosion pit, not necessarily a measure of the reduction of sectional area.
(b) Corrosion pits were up to 0·018 in. deep.
(c) There was no evidence of corrosion cracking.
(d) The ultimate tensile strength was in every case above 95 tons per sq. in. (up to 105 tons per sq. in.) and, consequently, within Specification.
(e) Bend tests in accordance with B.S. 2691 were carried out and gave satisfactory results although inferior to results for new wire.
(f) It was considered by one tester that due to the incidence of corrosion pits over half the circumference as mentioned above, the corrosion had resulted from condensation in the enclosed air space in the Freyssinet cable during shipping and storage, the non-pitted zones then corresponding with the portion of the outer surface in contact with the bituminous paper.

(6) Fig. 2 shows a photograph of three samples of wire with a magnification of about 2. These samples were chosen to illustrate the worst pitting and the lowest sample indicates the 'zoning' of corrosion found on some samples.

(7) Fig. 3 is a photograph of a prepared and etched microsection of the steel at and just below the surface with a magnification of about 60. The black top half is the Bakelite mount and the bottom half the steel. The surface roughness caused by corrosion and pits is evident.

Fig. 2. (Paper A.)

Fig. 3. (Paper A.) × 60

(8) Notwithstanding the apparently satisfactory results of static tensile strength, it was considered by the Consultants responsible that the degree of pitting on a good deal of the wire was such as to make it unsuitable for use and, accordingly, this wire was replaced. It can be stated at this stage that this decision was substantiated by the results of tests subsequently initiated by the author's firm (who were called in subsequently) and reported below. It will be appreciated that this called for the most painstaking visual inspection of all wire by the Consultant's Resident Engineering staff and that this type of procedure has many drawbacks.

Test on Fatigue Resistance

(9) Six 3 ft. long samples of 0·276 in. diameter wire were taken from the site in Lagos and were chosen so as to exhibit various degrees of pitting. The worst depth of pitting was up to 0·05 in. and a description of the degree of pitting in each sample was as follows :—

Sample No.	Appearance of Surface
1	Pitting extensive over most of surface.
2	Pitting moderately severe over ½ of surface on one side.
3	Slight pitting over about ½ of surface on one side.
4	As sample 1.
5	Slight pitting over most of surface.
6	No significant pitting.

(10) In addition, a specimen was taken from new wire from the then current production run in the United Kingdom. This was specimen No. 7 which was 'Bright' (i.e., not rusted at all).

(11) All samples were first cut to provide specimens for static strength tests and specimens for fatigue tests. The results of the static strength tests were as follows :—

Sample No.	Breaking Stress (Tons per sq. in.)	R.A. %
1	98·51	36
2	108·96	34
3	105·97	33
4	109·70	36
5	111·57	33·5
6	103·73	27·5
7	107·1	21·5

(12) For the fatigue tests the specimens were tested in an Amsler vibrophore at a mean stress of approximately 60 tons per sq. in. with a variation of + or — 10 tons per sq. in. The actual values of stress were as follows.:—

Mean Stress	Maximum Stress	Minimum Stress
59·93	69·81	50·04

(13) All samples were tested at a frequency of 138 cycles per second. The following were the results :—

Sample No.	Cycles $\times 10^{-6}$	Point of failure
1	0·390	Just in free test length
2	1·450	In free test length
3	0·618	" " " "
4	0·561	" " " "
5	1·319	" " " "
6	8·975(unbroken)	—
7	10·879(unbroken)	—

All samples which broke failed with typical fatigue fractures showing no reduction of area at the fracture.

Discussion of Results

(14) The tests reported in this Paper showed conclusively that there was a significant difference between the pitted wires and unpitted wires in so far as their resistance to fatigue loading was concerned.

(15) Whilst it could be argued—and is accepted—that the stress variations imposed on the test specimen (± 16%) were considerably greater than those normally encountered in wires in actual prestressed concrete structures, the purpose of the test was to establish only whether there was a difference between the two sets of wire and to that extent the tests were entirely successful.

(16) The actual stress variations in wire in prestressed concrete structures would be quite small, often between 3% and 10% (+ and —). As the differences in fatigue resistance for the tests reported herein were so marked, it was thought that even for normal stress variations corrosion might, under certain circumstances leading to extensive pitting, significantly affect the fatigue resistance of the wire.

(17) The results of controlled tests reported in Paper B 'straddle' the results given in this paper (excepting the depth of pitting the reported value of which for the Lagos wire could not be reproduced in the U.K. Tests). It can be seen from Fig. 6 of that paper however, that for normal stress variations in normal atmospheric conditions it is quite unlikely that the fatigue resistance would be lowered by corrosion to an extent which would require special consideration.

(18) However, this aspect of corrosion should not be overlooked when work is carried out on particularly corrosion-prone sites (of which Lagos is one). Some of the points to be considered are given in the conclusions below.

Conclusions

(19) It has been previously established that general light corrosion of bright wire is not necessarily adverse to the performance of the wire since, for example, bond characteristics are improved. It is also interesting that from Fig. 4 in Paper B it can be seen that the surface characteristics appear to be improved by this initial light general corrosion.

(20) Quite heavy corrosion and pitting did not appear to affect the wire characteristics—other than fatigue resistance—to a degree whereby they fell below Specification.

(21) Where work is carried out in climates which give rise to a particular corrosion risk, this aspect should be carefully considered so that any significant pitting of wires prior to their incorporation in the works is avoided.

(22) Wire used in such climates (e.g. the tropics) should not be made up into cables prior to shipping.

(23) Wire shipped in coils should have a very robust wrapping of bituminous or other similar material within which wrapping some rust inhibitor could be used with advantage ; the wires subsequently being cleaned.

(24) Immediately on arrival on site following shipping and transport, the wire should be unwrapped and inspected generally to see whether any pitting or the like has taken place.

(25) Until actually incorporated in the works, the wire should be stored on site in a manner which inhibits further corrosion or pitting. On particularly bad sites, such as e.g. Lagos, where there is salt spray in the atmosphere, wire could be stored within air conditioned and dehumidified sheds. Alternatively polythene wrapping with suitable dehumidification might be resorted to.

(26) General reference to pitting in a specification is not entirely adequate since pitting can be a matter of degree and almost all wire will show pitting when viewed through adequate magnification.

(27) Similarly, with present data on this subject, detailed inspection by site staff to check on the degree of pitting can be a difficult operation and measures should be taken, if at all possible, to avoid the necessity therefor.

(28) Further research is required to ascertain what degree of pitting is acceptable so that a clear yardstick can be provided for personnel inspecting wire on an 'accept/reject' basis. Such research should be aimed at providing the simplest rules possible incorporating both the depth and general size characteristics of pitting and the incidence thereof.

Acknowledgements

(29) Messrs. Sandberg, Richard Johnson & Nephew, British Ropes and Messrs. Arthur Lee & Sons carried out the tests summarised in the first section of this paper. Messrs. Richard Johnson & Nephew carried out the tests described in the second section. Fig. 2, and Fig. 3 are published by courtesy of British Ropes Ltd., and Messrs Richard Johnson & Nephew Ltd., respectively. The assistance of the above-mentioned firms is gratefully acknowledged.

PAPER B.

NOTES ON SOME PUSH-PULL ALTERNATING STRESS TESTS ON CORRODED COLD DRAWN STRESS RELIEVED WIRE FOR PRESTRESSED CONCRETE

(1) A prior report * on the examination of some corroded wires from unused Freyssinet cables included some limited results of alternating stress tests in tension. These results led to the present experimental work since it was felt to be desirable to explore in more detail the effects of corrosion carried out under known or controlled conditions.

(2) Since the specific mechanism of corrosion in the Lagos cables was not accurately known and the chance of exactly reproducing the type and amount of pitting encountered rather remote, it was decided to expose similar ·276 in. diameter wire samples to (a) industrial air atmospheric corrosion and (b) to a salt spray environment for varying and known periods of time and then to subject the corroded wires to push-pull alternating tensile stresses.

(3) The wire used in the present tests was random selected eight foot diameter coil of ·276 in. diameter wire made to the requirements of British Standard 2691 : Part 1. It was cold drawn patented wire and had been straightened and stress-relieved and payed out straight when uncoiled.

(4) The chemical analysis of the coil was Carbon ·78, Silicon ·18, Sulphur ·036, Phosphorus ·02, Manganese ·67, Nickel ·047, Copper ·128, Chromium ·069.

(5) The coil was cut into consecutively numbered 25 foot lengths and checked along its length for uniformity of properties with the results shown in Table 1.

Table 1

Uniformity check tests on tensile and elastic properties.

Length No.	Tensile lb.	Tensile ton/sq. in.	Proof Stress tons/sq. in. ·1%	·2%	Elongation % on 10*D*
1	14090	105·1	91·8	94·8	5·7
15	14000	104·5	91·6	94·0	5·7
30	14050	104·8	91·0	94·2	5·7
45	14050	104·8	90·5	93·4	5·7
59	14120	105·4	92·1	95·2	5·7
	D = nominal ·276 inch.				

(6) Wire lengths 1-10 were cut into 13 in. lengths and again numbered consecutively. The centre six inches of each was very lightly polished in a lathe using 000 emery paper to remove traces of rust. Both ends were spiral cold rolled for a length of four inches with an Amsler spiral roller tool so as to prepare them prior to exposure for the subsequent fatigue tests. A length of four inches at both ends was then twice dipped in clear cellulose varnish leaving the centre five inch portion clean and grease free after a final wipe with trichlorethylene.

(7) These prepared numbered specimens were then carefully loaded in a series of perspex frames and so arranged as to give similar exposure to all wires in the frame.

* Paper by A. Goldstein.

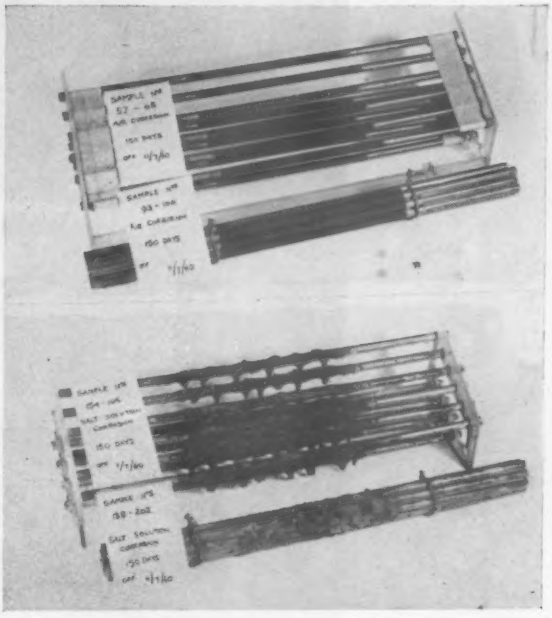

Fig. 1. (Paper B.) — Two typical frames with corroded wires after 150 days air and 150 days salt spray exposure.

(8) The frames of wires for air exposure were placed on a raised wooden plinth on a flat roof adjacent to the Manchester Bradford Road gas works facing in an East-West direction. The typical degree of pollution is shown in Table 4. Similarly the frames of wires for the continuous salt spray corrosion were loaded into a standard stainless steel cabinet operated by filtered controlled pressure air and continuous checks for mist volume being made using mist collectors inside the cabinet. The salt solution used contained 50 gms. per litre of a high grade dairy salt. A rainfall record was also kept for the period of the outdoor wire exposures which began in February 1960. This averaged 1·2 mm. per day for 150 days and the highest daily figure was 17 mm.

(9) After exposure and at intervals of 7, 25, 50, 100 and 150 days one frame of wires was withdrawn from both the roof and salt spray stations to give sets of corroded wires for tensile and fatigue tests.

(10) Photographic records were made of the appearance of the wires immediately after each exposure period. Fig. 1 shows a pair of such frames after 150 days in air and salt spray.

(11) The appearance after cleaning typical wires of both series in highly inhibited hydrochloric acid for typical wires is shown in Figs. 2 and 3.

(12) A wire from each ' air ' and ' salt spray ' frame was carefully cleaned to remove all adherent corrosion products and a two inch length cut from it and weighed. From these weights the loss of area was calculated and the results are shown in Table 2.

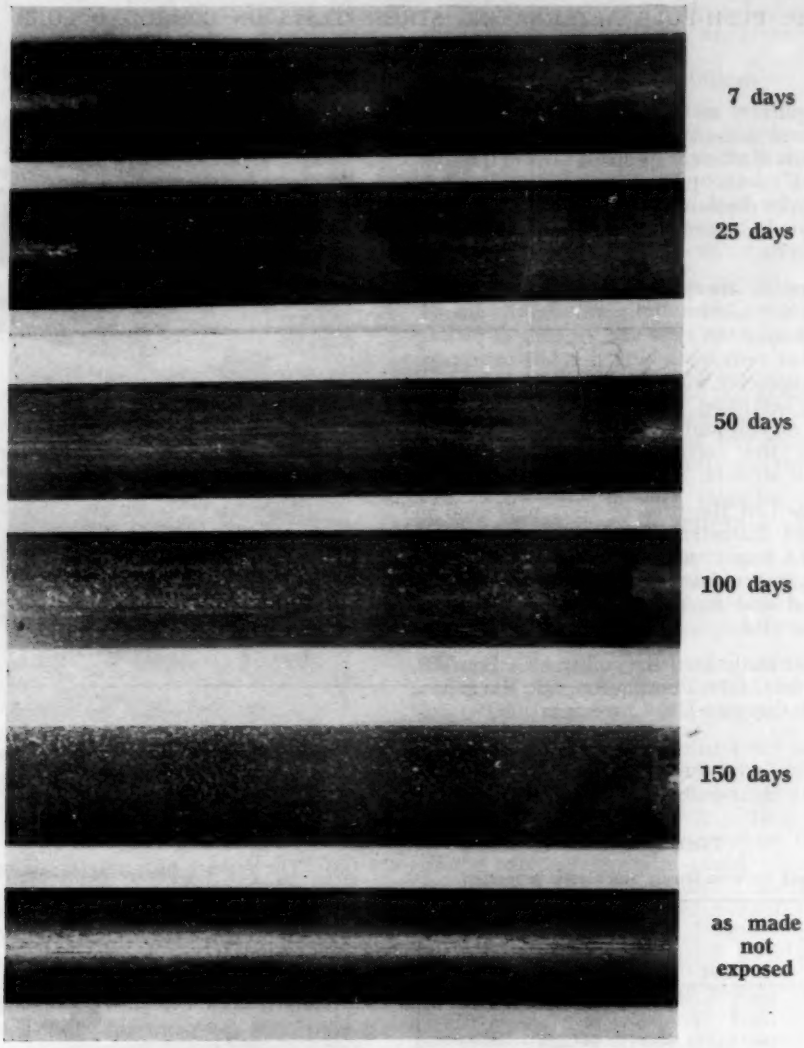

Fig. 2. (Paper B.) Air corroded wires — rust removed.

Table 2

Loss in area due to corrosion based on weight tests.

Corrosion medium	AIR	Salt-Spray
Exposure Period	% Area Total Loss	% Area Total Loss
7 days	·297	1·085
25 days	·638	1·745
50 days	1·043	4·003
100 days	2·047	8·368
150 days	2·592	14·324

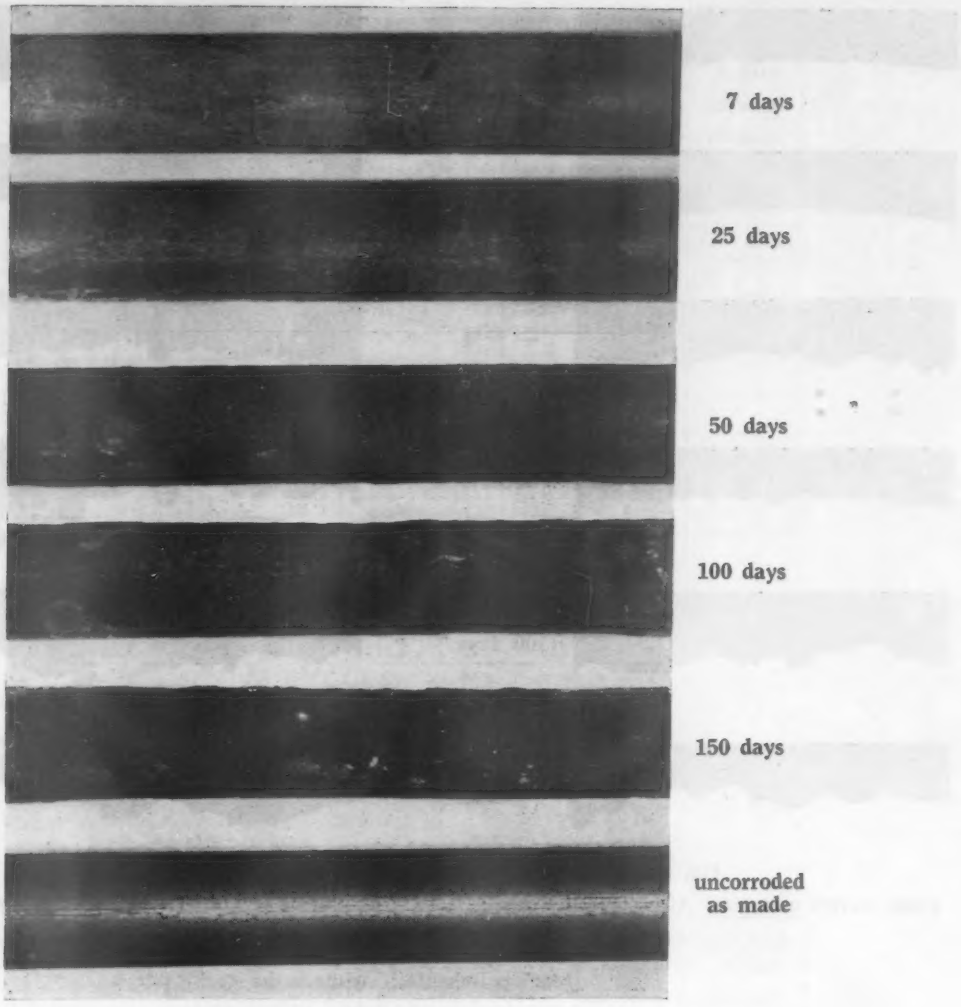

Fig. 3. (Paper B.) Typical salt spray corroded wires — rust removed.

Table 3

Tensile values of wires from each frame after various exposure times.

All tests were made simultaneously 250 days after commencement of tests.

"AIR CORROSION"

Exposed Time	7	25	50	100	150	days
Tensile	13950	14000	13750	13850	13500	lb.
Tensile	104·1	104·5	102·6	103·3	100·7	tons/sq. in.*

"SALT SPRAY" CORROSION

Exposed Time	7	25	50	100	150	days
Tensile	13800	13600	13350	12650	11600	lb.
Tensile	103·0	101·5	99·6	94·4	86·6	tons/sq. in.*

* Calculated on original ·276 diameter.

Table 4

Typical analysis and quantity of atmospheric deposits in the atmospheric exposure test zone.

pH of solubles	4-4·5	Tons/sq. mile/month
Soluble chlorides reported as *Cl*	·75	" " " "
Soluble sulphates reported as SO_4	3·00	" " " "
Total dissolved matter	11·40	" " " "
Insoluble tarry matter	·40	" " " "
Insoluble carbonaceous matter	1·00	" " " "
Insoluble ash	15·00	" " " "

(13) Tensile tests were also carried out from each set of exposed wires with the results recorded in Table 3.

(14) As a means of assessing and recording the degree of corrosion and pitting, cleaned wires were bedded in black Bakelite and cross sections prepared by diamond polishing. Typical average surface zones are shown as two series of photographs at 60 magnifications in Figures 4 and 5.

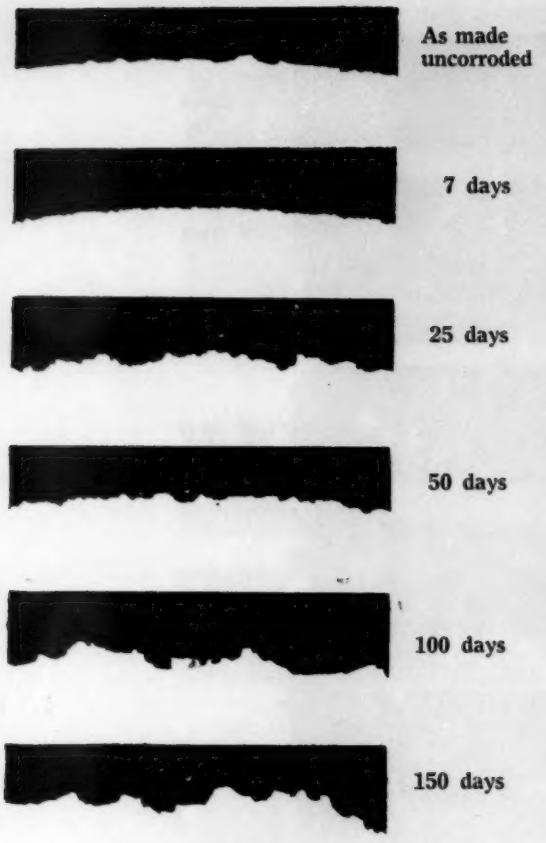

Fig. 4.
Cross section of typical Air corroded wires × 60

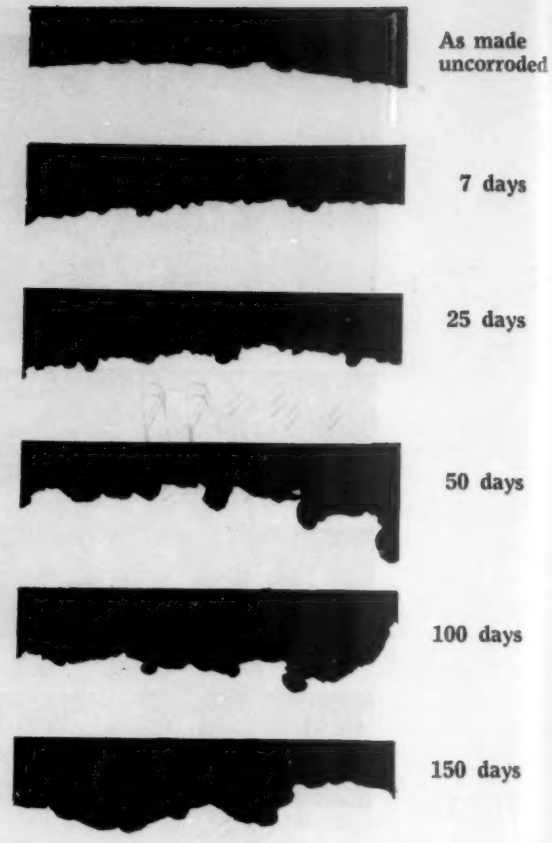

Fig. 5.
Cross section of typical Salt Spray corrosion × 60

(Black is Bakelite ; white is the steel.)

(15) The remaining sets of corroded wires were in turn tested on a 10 ton Amsler Vibrophore push-pull fatigue testing machine. A constant average tensile stress of 60 tons was employed and the results obtained are shown in the curves of Figs. 6 and 7, the stress being calculated in all cases on a nominal diameter of ·276. For the type of test carried out the scatter of results was surprisingly small.

Conclusions

(16) The results of the push-pull fatigue tests reported indicate that for periods up to 25 days in air there is either an improvement or practically no deterioration in the fatigue value for 10 million cycles unbroken when compared with the new unused wire and that the value is higher than usually obtained in practical structures with live loads applied.

(17) For salt spray corrosion the results indicate a progressive falling off with exposure time, and although in practice no wire for prestressing would ever be subjected to such treatment in the normal way, the results are of interest.

(18) The overall implication of the data is that cold drawn patented wire for prestressing work with a uniformly 'air' corroded surface can be safely used provided pitting has not become serious and in fact the curves of Fig. 6 show that some weathering raises the limiting fatigue stress above that of the new wire direct from the factory.

(19) Although it is recommended that due consideration is given to packaging for each specific case where overseas shipment is involved, it is reassuring to record that the difficulties reported on the Lagos cables are, so far as the present author is aware, a special and isolated case which has not been met elsewhere in the past 15 years of world wide experience.

Discussion

The Council would be glad to consider the publication of correspondence in connexion with the above papers. Communications on this subject intended for publication should be forwarded to reach the Institution by January 31st, 1962.

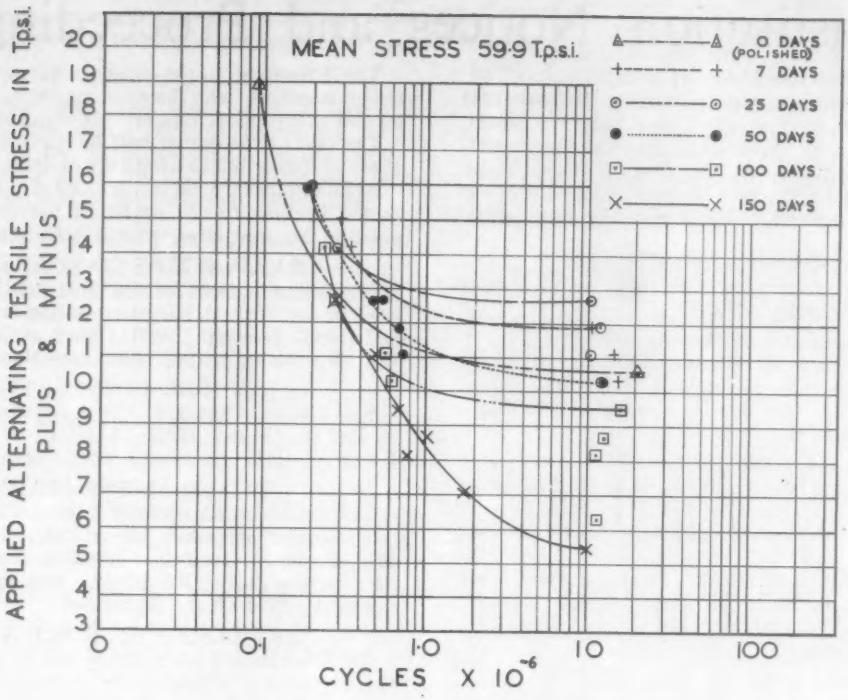

Fig. 6.—Atmospheric corrosion

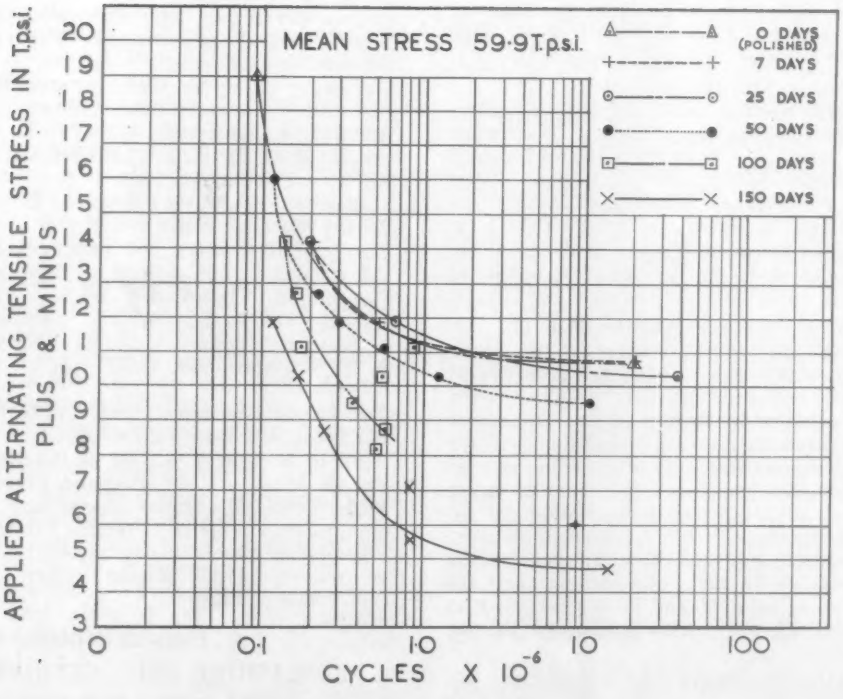

Fig. 7.—Salt spray corrosion

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 5th October, 1961, at 6 p.m., when Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., will be installed as President for the Session 1961-62 and will give the Presidential Address.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 12th October, 1961.

Ordinary Meeting at 6 p.m., when a paper on "Professional Practice and Drawing Office Procedure" will be given by Mr. L. R. Creasy, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E.

Thursday, 26th October, 1961.

Ordinary General Meeting for the election of members at 5.55 p.m. This meeting, which is open only to corporate members of the Institution, will be followed by an Ordinary Meeting at 6 p.m., when a paper on "The Plastic Design of Latticed Portal Frames" will be given by Mr. B. H. Fisher, B.Sc., A.M.I.Struct.E., A.M.I.C.E., Dr. Jacques Heyman, M.A., A.M.I.C.E., and Dr. L. G. Jaeger, M.A., A.M.I.C.E.

Thursday, 9th November, 1961.

Joint Meeting of the Institution of Structural Engineers and the Institution of Royal Engineers at 6 p.m., when Brigadier H. A. T. Jarrett-Kerr, O.B.E., will give a paper on "The Development of Engineer Equipment for the Army." Advance copies may be obtained on application to the Secretary.

Thursday, 23rd *November*, 1961.

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a paper on "Services and Air Conditioning for Tall Buildings" will be given by Mr. J. R. Kell, M.I.Mech.E., P.P.I.H.V.E., F.Inst.F., M.Cons.E.

Thursday, 14th December, 1961.

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a paper on "The Chemical and Physical Effects of Aggressive Substances on Concrete" will be given by Dr. P. E. Halstead, B.Sc., F.R.I.C.

Members wishing to bring guests to the Joint Meeting and the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

A MESSAGE FROM THE PRESIDENT TO STUDENTS AND GRADUATES

The President would wish to draw the attention of younger members, both students and graduates, to the advantages to be gained by their attending the regular meetings of the Institution either in London or in the Branches. All these meetings are open to them and are not restricted to older members. It is his experience that he never attends a meeting without adding to his store of information and to his practical experience. He invites, therefore, and, indeed, would urge all the younger members to do their utmost to attend the meetings as often as possible, and he is confident that by so doing they will improve their prospects of passing their examinations.

MAITLAND TESTIMONIAL DINNER

The Testimonial will be presented to Major Maitland at a dinner at the Dorchester Hotel on Thursday, 2nd November, at 7.00 for 7.30 p.m.; dress, lounge suits.

The President hopes that as many as possible of those members who have contributed to the Testimonial will support him on this occasion.

The cost of the dinner will be £2 10s. 0d, including wines at dinner but not liqueurs or drinks before dinner.

Applications for tickets, accompanied by remittances, should be sent to the Assistant Secretary as soon as possible, and not later than 19th October.

EXAMINATIONS, JANUARY, 1962.

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 9th and 10th January, 1962 (Graduateship) and 11th and 12th January, 1962 (Associate-Membership).

ANNUAL DINNER, 1962.

The Annual Dinner of the Institution will be held at the Dorchester Hotel, London, W.1., on Friday, 4th May, 1962.

SESSIONAL PROGRAMME

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1962-63. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

ARTICLES FOR PUBLICATION

The Literature Committee would be glad to consider short articles of a descriptive character for publication in the Journal. The object of such articles would be to describe the difficulties encountered during construction and the ways in which they were overcome, and to set out details of structures successfully erected. Articles should be accompanied by good illustrations.

LECTURES ON ARBITRATION

A series of lectures on Arbitration has been arranged by the Institute of Arbitrators. The programme is primarily intended for members of the principal professional bodies, in collaboration with which the lectures have been arranged. The programme is as follows :—

Tuesday, 10th October, 1961. ARBITRATION AGREEMENT
Nature of Arbitration; Arbitration Act 1950; Arbitration Agreement. By J. R. W. Alexander, C.B.E., M.A., LL.B., F.C.I.S., F.I.Arb.

Tuesday, 17th October, 1961. DISPUTE
Dispute; Arbitrator; Umpire; Expert Witness. By A. B. Waters, M.B.E., G.M., F.R.I.B.A., F.I.Arb.

Tuesday, 24th October, 1961. PRELIMINARY PROCEEDINGS
Preliminary Proceedings; Pleadings; Evidence; Case Stated; High Court. By Lord Meston.

Tuesday, 31st October, 1961. HEARING AND AWARD
Hearing; Award; Costs; Enforcement. By J. R. W. Alexander, C.B.E., M.A., LL.B., F.C.I.S., F.I.Arb.

Tuesday, 14th November, 1961. PRACTICE ARBITRATION
For those attending the Lectures.

All to be held at 6 p.m. at the Royal Institute of British Architects, 66, Portland Place, London, W.1. There is no admittance charge and tickets are not required except for the Practice Arbitration.

Enquiries should be addressed to :—The Institute of Arbitrators, 27 Regent Street, London, S.W.1. (Tel : Regent 6426).

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

The opening meeting of the Session will be held on Tuesday, 17th October, 1961, when Mr. J. B. Story,

M.I.Struct.E., will give the Chairman's Address. The meeting will commence at 7 p.m., and will be preceded by a Hot Pot Supper. A coach will run from Liverpool.

Monday, 13th *November*, 1961.

Joint Meeting with the Manchester Society of Architects. "Architectural Misconceptions of Engineering," by Mr. A. J. Harris, B.Sc., M.I.Struct.E., M.I.C.E., at 6.30 p.m. N.B. Not Tuesday, 14th November, as printed in the Sessional Programme.

Wednesday, 6*th December*, 1961.

"The Plastic Design of Steel Framed Structures," by Professor M. R. Horne, M.A., Sc.D., Ph.D., A.M.I.C.E., at 6.30 p.m.

Meetings will be held at the College of Science and Technology, Manchester, and will be preceded by light refreshments.

MERSEYSIDE PANEL

Monday, 20*th November*, 1961.

"Economics of Framed Structures," by Mr. L. R. Creasy, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., at Liverpool University, in the New Civil Engineering Building, Brownlow Hill.

Wednesday, 13th *December*, 1961.

"The Construction of a Large Covered Reservoir," by Mr. J. W. Baron, M.I.Struct.E., at the College of Building, Clarence Street, Liverpool.

Meetings will commence at 6.30 p.m., and will be preceded by light refreshments.

Joint Hon. Secretaries: W. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs., and M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

The Annual Dinner and Ladies Evening will be held at the Botanical Gardens, Birmingham, on Saturday, 14th October, 1961.

A Meeting of the Branch will be held at the Byng Kenrick Suite, College of Advanced Technology, Gosta Green, Birmingham, on Friday, 27th October, 1961, at 6.30 p.m., when Mr. J. E. C. Farebrother, M.I.Struct.E., will give the Chairman's Address, "Concrete—Past, Present and Future." Tea will be served from 5.00 to 5.45 p.m.

Friday, 1*st December*, 1961.

"The Structural Design of a New Store for Rackhams (Harrods) Ltd. in Birmingham," by Mr. F. G. Coffin, A.M.I.Struct.E., and Mr. J. N. Martin, A.M.I.Struct.E., A.M.I.C.E., at the Byng Kenrick Suite, College of Advanced Technology, Birmingham, at 6.30. p.m. Tea will be served from 5.00 to 5.45 p.m.

Tuesday, 5*th December*, 1961.

Joint Meeting with the East Midlands Local Association of the Institution of Civil Engineers, at the East Midlands Electricity Board Demonstration Theatre, Irongate, Derby, at 6.15 p.m.: "Vibration Problems in Relation to Foundation Design and Construction," by Mr. J. H. A. Crockett, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.C.E.; refreshments from 5.30 p.m.

Hon. Secretary: A. N. Jones, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.Mech.E., 4 Sunny Bank Road, Quinton, Birmingham, 32.

GRADUATES' AND STUDENTS' SECTION

Friday, 3*rd November*, 1961.

"Proposed Bridge Over the English Channel," by Dr. A. R. Flint, B.Sc., at the Engineering Centre, Birmingham, at 6.30 p.m.; tea from 6 p.m.

Wednesday, 15*th November*, 1961.

"Elevated Roadways," by Mr. G. B. Smedley, B.Sc.(Tech.).A.M.I.Struct.E., A.M.I.C.E., at the Electricity Demonstration Hall, Irongate, Derby, at 6.15 p.m.; tea from 5.30 p.m.

Friday, 8*th December*, 1961.

"Recent Research in Timber Engineering," by Mr. I. D. G. Lee, B.Sc.(Eng.), at the Engineering Centre, Birmingham, at 6.30 p.m.; tea from 6 p.m.

Hon. Secretary: H. T. Dodd, A.M.I.Struct.E., Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwicks.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

A meeting will be held at the Cleveland Scientific and Technical Institution, Middlesbrough, on Thursday, 12th October, 1961, when an Address will be given by the Chairman-Elect, Mr. L. Dobson, M.I.Struct.E.

A meeting will be held at the Neville Hall, Newcastle, on Monday, 16th October, 1961, when the President of the Institution, Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., will install the Branch Chairman, Mr. L. Dobson, M.I.Struct.E., who will then give the Chairman's Address.

Wednesday, 1*st November*, 1961.

At Newcastle. "The Volta Bridge," by Mr. C. R. Blackwell, B.Sc.,

Tuesday, 7*th November*, 1961.

At King's College, Newcastle. "What is Structural Engineering?", by Mr. P. D. Newton, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E.

Tuesday, 21*st November*, 1961.

At Middlesbrough. Joint Meeting with the Tees-Side Branch of the Northern Architectural Association. "Transport in Urban Areas," by Lt.-Colonel C. D. Buchanan, B.Sc.(Eng.), A.C.G.I., A.R.I.B.A., A.M.I.C.E., A.M.T.P.L.

Tuesday, 28*th November*, 1961.

At Sunderland Technical College. "Reinforced Concrete Bridges," by Mr. L. Dobson, M.I.Struct.E. (Branch Chairman).

Tuesday, 5*th December*, 1961.

At Middlesbrough. "The Runcorn-Widnes Bridge," by Mr. J. K. Anderson, M.A., M.I.C.E.

Wednesday, 6*th December*, 1961.

At Newcastle. "Some New Developments and Applications of Prestressed Concrete," by Mr. H. Kaylor, (Tech.), M.I.Struct.E., M.I.C.E.

Meetings will commence at 6.30 p.m., preceded by buffet tea at 6 p.m. Unless stated otherwise, meetings in Newcastle will be held at the Neville Hall, and those in Middlesbrough at the Cleveland Scientific and Technical Institution.

Hon. Secretary: P. D. Newton, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., 6, Cornfield Road, Linthorpe, Middlesbrough, Yorks.

NORTHERN IRELAND BRANCH

The opening meeting of the Session will be held on Tuesday, 31st October, 1961, when a paper on "The Structural Design of Merchant Ships" will be read by Mr. J. Williamson, B.Sc., A.M.R.I.N.A.

Tuesday, 5th December, 1961.

"Model Methods," by Professor S. R. Sparkes, M.Sc., Ph.D., M.I.Struct.E., M.I.C.E.

Both meetings will be held in the David Keir Building, Civil Engineering Department, Queen's University, Belfast, and will commence at 6.30 p.m., preceded by tea from 5.45 p.m.

Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast.

SCOTTISH BRANCH

The opening meeting of the Session will be held on Monday, 9th October, 1961, when Mr. K. D. Mathewson, B.A., M.I.Struct.E., M.I.C.E., will give the Chairman's Address.

Tuesday, 10th *October*, 1961.

The Annual Dinner and Dance at the Grosvenor Restaurant, Gordon Street, Glasgow, at 6.30 p.m.

Wednesday, 15th November, 1961.

Joint Meeting with the West of Scotland Branch of the Institute of Welding. "Who Teaches the Science of Welding?" by Dr. J. Orr, B.Sc., M.I.Mech.E., M.I.E.S., at 7 p.m., preceded by light refreshments at 6.30 p.m.

Tuesday, 5th December, 1961.

"Crane Erection Problems," by Mr. W. H. Arch, B.Sc., A.M.I.C.E.

Friday, 15th December, 1961.

Joint Meeting with the Glasgow and West of Scotland Association of the Institution of Civil Engineers. "Cable Spinning on the Forth Road Bridge," by Mr. H. Shirley Smith, O.B.E., B.Sc., M.I.C.E.

Unless stated otherwise, meetings will be held in the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, and will commence at 7 p.m.

EDINBURGH SECTION

Wednesday, 1st November, 1961.

Joint Meeting with the Edinburgh and East of Scotland Association of the Institution of Civil Engineers, at the North British Station Hotel, Edinburgh, at 6 p.m. "Cable Spinning on the Forth Road Bridge," by Mr. H. Shirley Smith, O.B.E., B.Sc., M.I.C.E.

Tuesday, 28th November, 1961.

Joint Meeting with the Edinburgh and East of Scotland Association of the Institution of Civil Engineers at the Heriot-Watt College, Edinburgh, at 6 p.m. "Piling" by Mr. W. G. N. Geddes, B.Sc., M.I.Struct.E., M.I.C.E.

Hon Secretary: W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTHERN BRANCH

The opening meeting of the Session will be held on Friday, 10th November, 1961, when Mr. J. R. Lowe, B.Sc., M.I.Struct.E., A.M.I.C.E., will give the Chairman's Address.

Friday, 1st December, 1961.

"Instability of Thin-walled Members," by Dr. P. S. Bulson, B.Sc., A.M.I.Struct.E., A.M.I.Mech.E.

Both meetings will be held in the Main Lecture Theatre, Lanchester Building, The University, Southampton, and will commence at 6.30 p.m., preceded by tea in the Refectory (West Site) from 6 p.m.

Hon. Secretary: A. P. K. Tate, B.Sc.(Eng.), Ph.D., A.M.I.Struct.E., Department of Civil Engineering, The University, Southampton.

WALES AND MONMOUTHSHIRE BRANCH

The opening meeting of the Session will be held in Cardiff on Tuesday, 24th October, 1961, when the President of the Institution, Mr. F. R. Bullen, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., will install the Branch Chairman, Mr. E. Hughes, M.I.Struct.E., who will then give the Chairman's Address.

Wednesday, 1st November, 1961.

The Chairman's Address will be repeated in Swansea.

Monday, 4th December, 1961.

At Cardiff. "Structural Steelwork at Spencer Works," by Mr. A. V. Hooker, M.I.Struct.E., M.I.C.E. (Past Chairman) and Mr. C. J. E. Morris, M.A., A.M.I.C.E.

Friday, 15th December, 1961.

At Swansea University. Joint Meeting with the South Wales and Monmouthshire Branch of the Institution of Civil Engineers. "The Design of Reinforced Concrete through the Use of Plastic Models," by Mr. B. W. Preece, B.Sc.

Meetings in Cardiff will be held at the South Wales Institute of Engineers, Park Place and those in Swansea at the Mackworth Hotel, unless stated otherwise, and will commence at 6.30 p.m.

Hon. Secretary: W. D. Hollyman, A.M.I.Struct.E., 41 Greenfield Avenue, Dinas Powis, Glamorgan.

WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held on Friday, 13th October, 1961, when Mr. Clifford E. Saunders, M.I.Struct.E., will give the Chairman's Address and the presentation of the Branch Prize and the Francis Memorial Prizes will take place.

Thursday, 9th November, 1961.

Joint Meeting with the South Western Association of the Institution of Civil Engineers. "Engineer Support to the Christmas Island Nuclear Tests of 1958," by Brigadier R. B. Muir, R.E., C.B.E., B.Sc., M.I.Struct.E., M.I.C.E., A.M.I.Mech.E.

Friday, 1st December, 1961.

Special Meeting to which kindred professional and technical bodies will be invited. "Long Span Suspension Bridges with reference to the Severn Bridge," by Mr. M. F. Parsons, B.Sc., A.M.I.C.E.

Meetings will be held at Queen's Building, University Walk, Bristol, the opening meeting and that on 9th November in the Small Lecture Theatre, the one on 1st December in the Large Lecture Theatre. They will commence at 6 p.m. and will be preceded by a light tea at 5.30 p.m.

Hon. Secretary: M. S. G. Cullimore, B.Sc., Ph.D., A.M.I.Struct.E., Queen's Building, University Walk, Bristol, 8.

YORKSHIRE BRANCH

The opening meeting of the Session will be held in the Department of Civil Engineering, The University, Woodhouse Lane, Leeds, on Wednesday, 18th October, 1961, when Mr. T. E. S. White, B.Sc., M.I.Struct.E., M.I.C.E., will give the Chairman's Address. The Address will be repeated at the Royal Victoria Hotel, Sheffield, on Thursday, 19th October, 1961.

Wednesday, 15th November, 1961.

"The Design of a Cantilever Stand for the Sheffield Wednesday Football Club," by Mr. H. C. Husband, B.Eng., M.I.Struct.E., M.I.C.E., M.I.Mech.E., Mr. T. Holmshaw, M.I.Struct.E., and Mr. H. C. English, B.Eng., M.I.Struct.E., A.M.I.C.E. In the Department of Civil Engineering, The University, Woodhouse Lane, Leeds.

Wednesday, 29th November, 1961.

" Air Conditioning and Services for Tall Buildings," by Mr. J. R. Kell, M.I.Mech.E., P.P.I.H.V.E., M.Inst.F., M.Cons.E. At the Royal Victoria Hotel, Sheffield.

Wednesday, 13th December, 1961.

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers and the Yorkshire and Lincolnshire Branch of the Institution of Highway Engineers. " The Doncaster Motorway—Reflections in Retrospect," by Mr. S. Maynard Lovell, O.B.E., T.D., M.I.C.E., M.I.Mun.E. At the Metropole Hotel, King Street, Leeds.

Meetings will commence at 6.30 p.m., and be preceded by buffet tea at 6.15 p.m.

Hon. Secretary : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

SOUTH AFRICA BRANCH

Hon. Secretary : E. B. Kretzschmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary : E. G. Bennett, P.O. Box 932, Durban.

Cape Section Hon. Secretary : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

SECTION NOTICES

AUCKLAND (NEW ZEALAND) SECTION

Hon. Secretary : A. Donald, B.Sc.(Hons.), A.M.I.Struct.E., 122, Matipo Road, Te Atatu, Auckland, New Zealand.

EAST AFRICAN SECTION

Hon. Secretary : K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

NIGERIAN SECTION

Hon. Secretary : A. Brimer, M.I.Struct.E., Brimer, Andrews & Nachshen, Private Mail Bag 2295, Lagos, Nigeria.

REPUBLIC OF IRELAND SECTION

Hon. Secretary : P. J. Carroll, M.E., A.M.I.Struct.E., A.M.I.C.E., A.M.I.C.E.I., 9, Laburnum Road, Clonskea, Dublin, Ireland.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon.Secretary : J. R. M. MacIntyre, A.M.I.Struct.E., c/o Redpath, Brown & Co. Ltd., P.O. Box 648, Singapore.

SOUTH WESTERN COUNTIES SECTION

The opening meeting of the Section will be held at the Grand Hotel, The Hoe, Plymouth, at 6.30 p.m., on Friday, 3rd November, 1961, when Mr. J. D. Norfolk, M.I.Struct.E., M.I.C.E., M.I.Mun.E., will give the Chairman's Address.

Friday, 8th December, 1961.

" Site Production and Placing of Concrete," by Mr. R. J. Tatt, A.M.I.C.E., of the Cement and Concrete Association, at the Duke of Cornwall Hotel, Plymouth, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : C. J. Woodrow, J.P., M.I.Struct.E., " Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

ADDITIONS TO THE LIBRARY

BRITISH IRON & STEEL RESEARCH ASSOCIATION. *Annual Report,* 1960, London. 1961.

CANADIAN INSTITUTE OF TIMBER CONSTRUCTION. *Timber Construction.* A Manual for Architects and Engineers. Ottawa, 1961. Presented by Mr. H. Feather.

COLUMN RESEARCH COUNCIL. *Guide to Design Criteria for Metal Compression Members.* Ann Arbor, Michigan, U.S.A., 1960, 319, West Engineering, University of Michigan. Presented by Mr. L. L. Kenchington.

CORLEY, W. G., SOZEN, M. A., and SIESS, C. P. *The Equivalent Frame Analysis for Reinforced Concrete Slab.* Univ. of Illinois Civil Eng. Studies Struct. Res. Series No. 218, 1961.

DEPARTMENT OF SCIENTIFIC & INDUSTRIAL RESEARCH. Forest Products Research Special Report No. 15. *Working Stresses for Structural Laminated Timber,* London, 1961.

Fedération et Syndicats des Industries Mécaniques et Transformatrices des Métaux Annuaire, 1960-1961 Paris, 1961 (Reference Library).

GAMBLE, W. L., SOZEN, M. A., and SIESS, C. P. *An Experimental Study of a Reinforced Concrete Two-way Floor Slab,* Univ. of Illinois Civil Eng. Studies Struct. Res. Series No. 211, 1961.

GRASSIE, J. C. *Applied Mechanics for Engineers,* London, 1960. Presented by Mr. L. M. Ramage.

Indian Roads Congress Road Research Bulletin No. 7, Report on the Experimental Performance of different types of Base Courses for Roads under adverse moisture conditions tested on the road test track at Majherhat, Alipore, Calcutta (1949-1958) New Delhi, 1960.

IWINSKI, T. *Theory of Beams.* The Application of the Laplace Transformation Method to Engineering Problems. London, 1958. Presented by Mr. L. M. Ramage.

KRETSIS, K. *Stress Distribution in Continuous Beams in the Neighbourhood of Internal Supports.* London University Thesis, 1961.

MATHESON, J. A. L. and FRANCIS, A. J. *Hyperstatic Structures, Vol.* 2, London, 1960. Presented by Dr. E. Lightfoot.

VAES, J. F., GUILLEMIN, C. and DESTAS, A. *Minéraux d'Uranium du Haut Katanga.* Les Amis du Musée Royal du Congo Belge, Tervuren (undated). Presented by Mr. G. Everwyn.

VENUAT, M. and PAPADAKS, M. *Contrôle et Essais des Ciments, Mortiers, Bétons.* Paris, 1961. Presented by Mr. P. J. Gerard.

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Book Reviews

International Association for Bridge and Structural Engineering Publications, Volume 20, 1960. (Zurich : Verlag Leeman, 1960). 9½ in. × 6½ in., 414 plus viii pp.

This twentieth volume of the " Publications " contains eighteen papers, twelve of which are in English, two in French and four in German. As in previous volumes, the titles and summaries of the papers are given in the three languages.

The contributions in English deal with :— A general analysis of elasto.plastic three-dimensional frames, analysis of curved lattices with generalized joint loadings, optimization of framing arrangements for large metal roof systems, curved edge disturbances in circular cylindrical shells, cold formed sections in Britain, riveted web connexions in bending, spherical domes under unsymmetrical loading, analysis of corner-supported grillages, the vibrations of massive foundations on soil, membrane stresses in hyperbolic paraboloid shells circular in plan, elasto-plastic analysis of an interconnected beam system, and abnormal loading on two types of short span bridge.

The two French papers deal with the general theory of the buckling of orthotropic rectangular plates and the report of the work of the 5th Meeting of the European Committee on Concrete. The German contributions are on theory of shells of any shape, calculation of groups of piles with a non-linear relationship between force and penetration, the relaxation of steel wires and the notch influence on the fatigue strength.

International Association for Bridge and Structural Engineering. Preliminary Publication of the Sixth Congress, Stockholm, 1960. (Zurich : Verlag Leeman, 1960). 9½ in. × 6½ in., 928 plus xvi pp., 391 figs., S. Fr. 70.

This publication includes the papers dealing with basic subjects that were selected for consideration at the 6th Congress. It is planned that the " Final Report " will contain papers dealing with practical applications and special problems together with the discussions at the vatious Sessions of the Congress and the summaries by the General Reporters including their recommendations for the future.

The present volume is divided into six sections. The first deals with general subjects relating to properties of materials and the development of methods of calculation for metal and reinforced concrete structures. This is followed by two sections on metallic structures dealing respectively with connexions including high strength bolts and then with steel frames, slabs and walls together with erection and safety precautions. Reinforced and prestressed concrete are treated in the next two sections, progress and failures in bridge building and saftey being considered in the first of these while the other section is concerned with prefabricated structures. The volume concludes with some papers dealing with improtant new developments, particularly in composite construction.

The comprehensive general reports in each section are provided in French, German and in English.